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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

HYDROSTATIC UPLIFT IN PERVIOUS SOILS

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To Be Presented May 2, 1928.

Synopsis

This paper describes an original research to determine the hydrostatic uplift on the base of a structure, founded on a pervious soil, when there is no loss of head due to velocity of flow. Engineers have not been in agreement on this question, some believing that the pressure due to the hydrostatic head acts on the full area of the base, and others that the pressure acts only on a portion of the area because the remaining portion is in intimate contact with the soil.

The paper records the results of 218 tests, not including trial tests, nor those made to standardize the apparatus. The soil materials used were sand, gravel, and clay. The tests were conducted with both low and high hydrostatic heads. The size of the base area, the head, and the unit loading on the soil were varied sufficiently to demonstrate that the results are independent of these factors. Shallow depth of soil under the test base, however, does affect the results.

The effective area, or that portion of the base on which the uplift acts, is stated in percentage of the total base area. The test results show that this effective area is approximately 90 to 100%, according to the test conditions.

The apparatus used and the methods of conducting the tests are fully described and illustrated.

Introduction

There is little information on record regarding hydrostatic uplift in pervious soils. Although some tests have been published,† a search revealed a paucity of data, but a desire for more knowledge on this subject.

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in **August, 1928.**

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† *Transactions*, Am. Soc. C. E., Vol. LXX (1910), p. 352.

Divergent opinions are held by engineers on the magnitude of the hydrostatic uplift force acting on the base of a structure built on pervious soils. When the head is relatively high, the resulting uplift becomes an important consideration in the design and construction of dams, piers, caissons, and similar structures.

Obviously, there is no hydrostatic uplift under a structure founded on rock when water cannot enter the joint between the base and the rock surface. However, should water reach the base of the structure through a fissure or water-bearing seam, there would be uplift on that portion of the joint to which water could obtain access. This uplift would act only on that portion of the base area where close physical contact with solid rock did not exist.

Engineers now give recognition to hydrostatic uplift, when making calculations for important dams founded on pervious soils, and various assumptions are made to insure the safety of the structure. Ordinarily, an engineer selects one of the following assumptions according to his idea of the conditions obtaining at the site:

1.—Allowance for uplift due to total head acting on the full width of base.
2.—Allowance for uplift due to total head at the up-stream edge, which is assumed to diminish uniformly to zero at the down-stream edge.
3.—Allowance for uplift due to a portion of total head (say, three-fourths, two-thirds, or one-half) at the up-stream edge which is assumed to diminish to zero at the down-stream edge.
4.—When the down-stream edge of the base is below tail-water level, the uplift at this edge is taken at tail-water head instead of at zero, as in Assumptions (2) and (3).
5.—When conditions are favorable for a tight joint at the up-stream edge, then uplift is assumed as constant over the width of the base due to the head at the down-stream edge.
6.—Modifications of these assumptions are made to comply with the designer's idea for safety, especially when drainage ducts are provided.

The question arises whether or not it is ultra-safe to assume the uplift pressure as acting on the total area of the base, both for uniform intensity of uplift and also for varying intensity due to a flow of water through the soil beneath the base. Some engineers have favored the hypothesis that water under head enters between the particles of a pervious foundation soil, and, in consequence, the uplift pressure acts only on that portion of the base which is exposed to water. Under this hypothesis, the remaining portion of the base area, being in direct contact with the particles of the foundation soil, is not subject to uplift because the contact, between base and soil, prevents the water from reaching this portion; and, it is claimed, the effective area would have some relation to the voids in the foundation material.

In the opinion of other engineers, uplift pressure acts on the total area of the base. Under this hypothesis, the uplift pressure acting on the base of a structure is, first, a direct hydrostatic pressure on the base area exposed to the water; and, second, an indirect pressure through the soil particles in contact with each other and with the base.

The writer undertook some experimental research work in order to obtain information regarding these hypotheses, and this paper records the results of the tests made.

Some of the smaller tests were made in the writer's office. In collaboration with Thomas R. Lawson, M. Am. Soc. C. E., apparatus was made from designs prepared by the writer, and set up in a laboratory of Rensselaer Polytechnic Institute, Troy, N. Y. Funds for the tests at Rensselaer were made available through the kindness of the Trustees from the Laflin Fund for research work. The writer also designed other apparatus, built at his own expense, which was erected in a laboratory of Stevens Institute of Technology, Hoboken, N. J. There were two series of experiments—one with low heads and one with high heads.

Low-Head Tests

Tests were made with low hydrostatic heads acting on a glass jar and on metallic cans, embedded in sand. At the instant the jar or can commences to rise from the sand, the hydrostatic uplift, acting on the bottom, just equals the weight of the jar or can, plus the sand friction against its side. It was necessary, therefore, to determine this sand friction, which was done as follows: The can to be tested was placed on a support which rested on the bottom

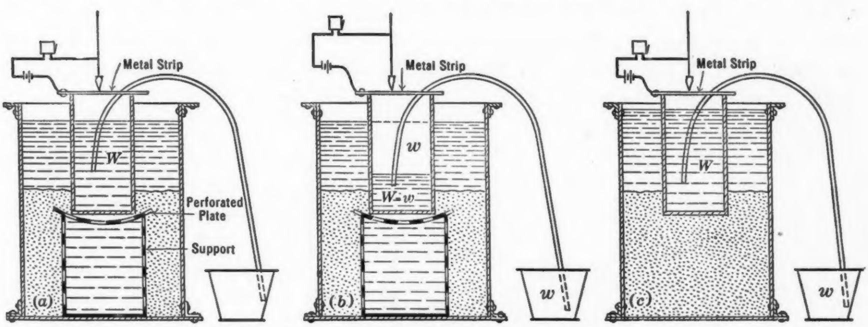

Fig. 1.—Sand Friction Tests.

of a containing vessel. This support had no top, and the can was placed on a perforated plate so that water could freely reach the bottom of the can. The supporting contact area between can and perforated plate was made small so as to be negligible (see Fig. 1 (*a*)). The support was filled with water, and wet sand was placed in the container to a predetermined height on the side of the can, and hand-packed. The can then was filled with water to give it additional weight, and afterward the container was filled with water to the same level as the water inside the can. The outside head was carefully measured from water surface to bottom of can. To indicate the first movement of the can, a plumb-bob was suspended just clear of a metal strip across the top of the can, and wired through a battery and a buzzer. The water inside the can was slowly siphoned until the can commenced to rise (Fig. 1 (*b*)). The water siphoned from the can was weighed, to determine the weight of water left in the can at the instant of movement.

This procedure is expressed mathematically as follows:

Let U = hydrostatic uplift on the bottom of the can.

C = weight of empty can and metal strip.

C_1 = weight of water having same volume as submerged material of can.

W = original weight of water in can.

w = weight of water siphoned until can starts to float.

F = sand friction against side of can.

Then,

$$U < W + C + F$$

After siphoning w at the instant the can begins to float,

$$U = W - w + C + F$$

Also,

$$U = W + C_1$$

Equating the values of U and solving for F:

$$F = w - (C - C_1)$$

in which, $(C - C_1)$ is the weight of the can in water.

Sand Friction Tests.—All tests in the writer's office were made with a glass jar. Ordinary silica sand was used, having a weight (dry) of 112 lb. per cu. ft., a specific gravity of 2.80, and voids, 35.5% by volume. The jar is illustrated in Fig. 2 (*a*). The container was a cylindrical glass aquarium, 8.69 in. in inside diameter by 14.75 in. in inside depth.

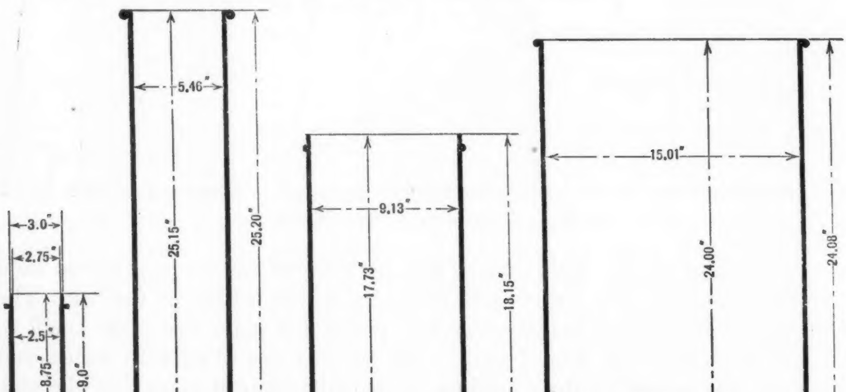

FIG. 2.

All tests with the metallic cans were made in the laboratory of Rensselaer Polytechnic Institute. The 9.22-in. can was made of tinned sheet metal, and both the 5.53-in. and the 15.09-in. cans were made of galvanized sheet iron.

The sand had a weight (dry) of 100.8 lb. per cu. ft., a specific gravity of 2.43, a fineness modulus of 2.09, and voids 33.5% by volume. The dimensions of the cans are shown in Fig. 2 (*b*), (*c*), and (*d*). Sand friction tests made with the 9.22-in. can in its original condition did not give results in harmony with the others, due to the lip formed by the side and bottom. The tests always showed greater friction and were rejected as unreliable. The bottom recess then was filled with plaster of Paris as shown in Fig. 2 (*c*), and the recorded tests were made with the can in this condition. The container for all the cans was a steel cylinder, 36 in. in inside diameter by 32½ in. in inside depth.

Twenty-nine sand friction tests were recorded, consisting of six with the 3-in. glass jar, six with the 5.53-in. can, eight with the 9.22-in. can, and nine with the 15.09-in. can. Many other tests were not recorded because they were considered doubtful due to faulty manipulation or conditions of can that precluded accurate results. The results of the tests did not show any material difference in friction due to the materials of which the cans were made, namely, glass, tin, or ganvanized iron.

After determining the friction, the result was reduced to pounds per square inch of side surface of can embedded in sand. The results, as averaged for similar conditions, are given in Table 1.

TABLE 1.—AVERAGES OF TWENTY-NINE SAND FRICTION TESTS.

Depth that can was embedded in sand, in inches.	Friction, in pounds per square inch of side surface of can in contact with sand.
0.5	0.01625
1.0	0.02338
2.0	0.03078
3.0	0.01616*
4.0	0.02636*
6.0	0.04700
8.0	0.04748
9.0	0.05655
12.0	0.05972
15.0	0.06790

* Doubtful.

The frictions were then platted on logarithmic paper (Fig. 3) and a mean line was drawn through the points, after omitting as doubtful the tests at 3-in. and 4-in. depths. The two discordant friction results were caused by a low one occurring at each depth, which made the averages too small. For all the uplift tests with the jar and cans, the friction was determined by reading the unit friction from the mean line (Fig. 3) and multiplying by the surface of the side in contact with the sand. This method eliminated slight irregularities due to manipulation. In order to find the mean line of friction (see Fig. 3), find the center of gravity, C, of all tests and the centers, A and B, above and below C, respectively. The mean line passes through these three points. Tests at 3-in. and 4-in. depths of the can embedded in sand were omitted as doubtful results. The formula for the mean line is:

$$\text{Friction, in pounds} = 0.0227\ (\text{depth embedded, in inches})^{0.3977}$$

Effective Area of Base.—A series of tests was made to determine the effective area of the base on which the hydrostatic uplift acted.

The jar or can was embedded to different depths in the same sand used for the friction tests. Each test was conducted in a manner similar to those for sand friction, the difference being that the can stood on the packed sand (Fig. 1(*c*)), instead of resting on a stand filled with water. Part of the water in the can was siphoned until the hydrostatic uplift just overcame the weight and sand friction.

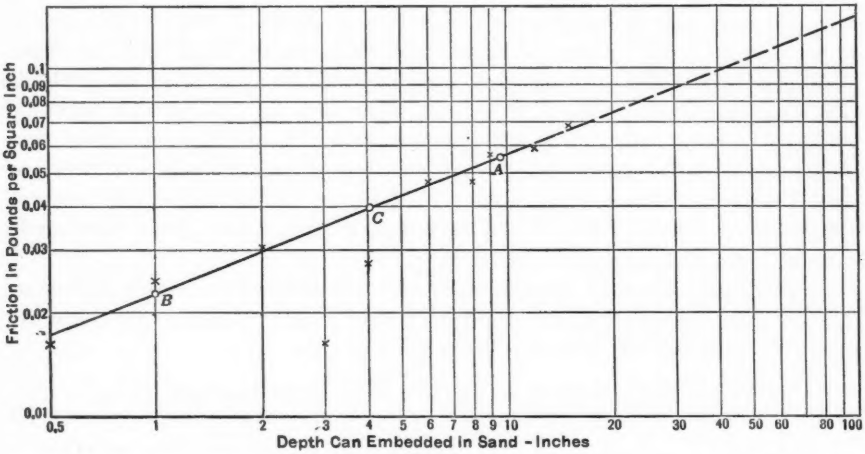

FIG. 3.

This procedure is expressed mathematically as follows:

Let A = outside area of base of can.
a = effective area of base on which hydrostatic uplift acts.
h = hydrostatic head on outside base of can.
W = weight of water in can.
w = weight of water siphoned until can just begins to float.
C = Weight of empty can and metal strip.
F = sand friction against side of can.

All measurements were in inches and pounds, and the weight of 1 cu. in. of water was taken at 0.0361 lb. Then:

$$\text{Hydrostatic uplift} = 0.0361 \times h \times a$$

and,

$$\text{Resisting forces} = W - w + C + F$$

When the uplift just overcomes the resistance, these two expressions are equal. As all values are determined by the test measurements, except a, the effective area of base can be calculated. Then:

$$\frac{a}{A} \times 100 = \text{percentage of base area on which hydrostatic uplift acts.}$$

Thirty-three tests were made with wet sand, hand-packed in the container, namely, eight with the glass jar, six with the 5.53-in. can, ten with the 9.22-in. can having bottom recess filled with plaster of Paris, and nine with the 15.09-in. can having the bottom reinforced on the inside with small angle-bars,

as it was discovered that the external hydrostatic pressure caused a deflection which was measureable.

Although an effort was made to secure similar conditions in each test, the variations in the results are probably due to manipulation and difficulty of accurately recording the low pressures.

Results of tests under similar conditions were averaged, and Table 2 gives the summary, with the tests arranged in order of depth that the can was embedded in the sand. It will be noted that with the low hydrostatic pressures used, the depth of the sand bed below the base of the cans did not seem to have any influence on the effective area.

TABLE 2.—EFFECTIVE AREA OF BASE WITH WET SAND PACKED IN CONTAINER.

Can tested.	Diameter of can, in inches.	Sand depth below base, in inches.	Depth can was embedded in sand, in inches.	Average effective area in percentage of base.
Glass jar	3.0	6.0	0.0	97.6
Glass jar	3.0	6.0	0.5	96.5
Glass jar	3.0	6.0	1.0	90.5
Glass jar	3.0	6.0	2.0	89.6
Metal can	15.09	7.5	3.0	98.2
Metal can	5.53	6.5	4.0	88.0
Metal can	9.22	11.0	4.0	99.7
Metal can	5.53	6.5	6.0	90.3
Metal can	9.22	11.0	6.0	86.7
Metal can	15.09	7.5	6.0	97.3
Metal can	5.53	6.5	8.0	81.8
Metal can	9.22	11.0	8.0	85.2
Metal can	9.22	11.0	9.0	82.5
Metal can	15.09	7.5	9.0	94.5
Metal can	15.09	7.5	12.0	92.7
Average of all tests				91.4
Average of tests omitting glass jar				90.6

A similar series of tests was made with the same sand hand-packed and tamped in a dry condition around the can. After filling the can with water, the container was filled on top of the dry sand to the same level and the test was conducted by siphoning water from the can as before. It was found that time had to be allowed for the water to saturate the sand, or the results would not be concordant.

The results are given in Table 3, being the averages of eighteen tests, namely, seven with the 5.53-in. can, six with the 9.22-in. can with plastered bottom, and five with the 15.09-in. can. The effective area in percentage is somewhat smaller than that obtained with the wet sand packing. It is possible that the results were affected by the time element mentioned.

HIGH-HEAD TESTS*

The apparatus consisted of a container made from a piece of extra heavy steel pipe, 12 in. in inside diameter, having a blind cast-iron flange screwed on the bottom. A cast-iron companion flange was screwed on the top, to which was bolted a specially designed cover fitted with a stuffing-box and packing gland. A solid steel contact rod, 3.99 in. in diameter by 60 in. long,

* Made in laboratory at Stevens Institute, Hoboken, N. J.

with the bottom end machined to a smooth surface at right angles to the axis, passed through the stuffing-box. The gland was packed with square-section steam piston-rod packing. There was an air-cock in the cover, and the container was tapped for a water-pipe connection and diametrically opposite for a pipe leading to a pressure gauge. The equipment is shown in Fig. 4. The contact rod weighed 216 lb., and had a base area of 12.53 sq. in. When supported by a soil, the unit load was, therefore, 17.24 lb. per sq. in., equivalent to 1.24 tons per sq. ft.

TABLE 3.—EFFECTIVE AREA OF BASE WITH DRY SAND PACKED IN CONTAINER.

Can tested.	Diameter of can, in inches.	Sand depth below base, in inches.	Depth can was embedded in sand, in inches.	Average effective area in percentage of base.
Metal can..........	5.53	6.5	4.0	90.1
Metal can..........	9.22	11.0	4.0	95.8
Metal can..........	5.53	6.5	6.0	88.5
Metal can..........	9.22	11.0	6.0	74.0
Metal can..........	15.09	7.5	6.0	93.9
Metal can..........	5.53	6.5	8.0	86.1
Metal can..........	9.22	11.0	8.0	67.2
Metal can..........	15.09	7.5	9.0	86.2
Metal can..........	15.09	7.5	12.0	88.2
Average of all tests	....	....	....	85.6

In these experiments, the soil to be tested was first put in the container and packed. The rod was then brought down into firm contact with the soil, and water was allowed to enter the container very slowly, while the air found exit through the cock in the stuffing-box flange. When the container was full of water and the air-cock closed, the pressure rose slowly. When the uplift was sufficient to balance the weight of the rod and to overcome the stuffing-box friction, the gauge pressure was recorded. This point was indicated by a buzzer wired to a plumb-bob and a copper strip lashed to the rod. The first movement of the rod could be detected before the buzzer sounded, by watching the light under the plumb-bob point. The device proved to be very sensitive.

Before each test, the difference in level between the base of the contact rod and the gauge center was measured. The gauge pressure, as registered, was corrected for this difference in head. The gauge also was calibrated before using, and care was taken to note that the contact rod was vertical by sighting along the plumb-bob cord.

Immediately after determining the hydrostatic uplift gauge pressure, a friction test was made. This was done by raising the contact rod just clear of the soil, so that its base was in complete contact with the water. The hydrostatic pressure required just to raise the rod was then recorded, as before.

Let A = base area of the contact rod.

a = effective area of rod, that is, the area on which uplift acts.

W = weight of rod.

F = friction of stuffing-box packing.

P = corrected hydrostatic gauge pressure when rod, resting on the soil, just commences to rise.

p = corrected hydrostatic gauge pressure when rod, being in complete contact with water, just commences to rise.

Then,

$$Pa = W + F = pA$$

or,

$$\frac{a}{A} = \frac{p}{P}$$

Then, the ratio of effective area to total base area, expressed in percentage, is $\frac{p}{P} \times 100$.

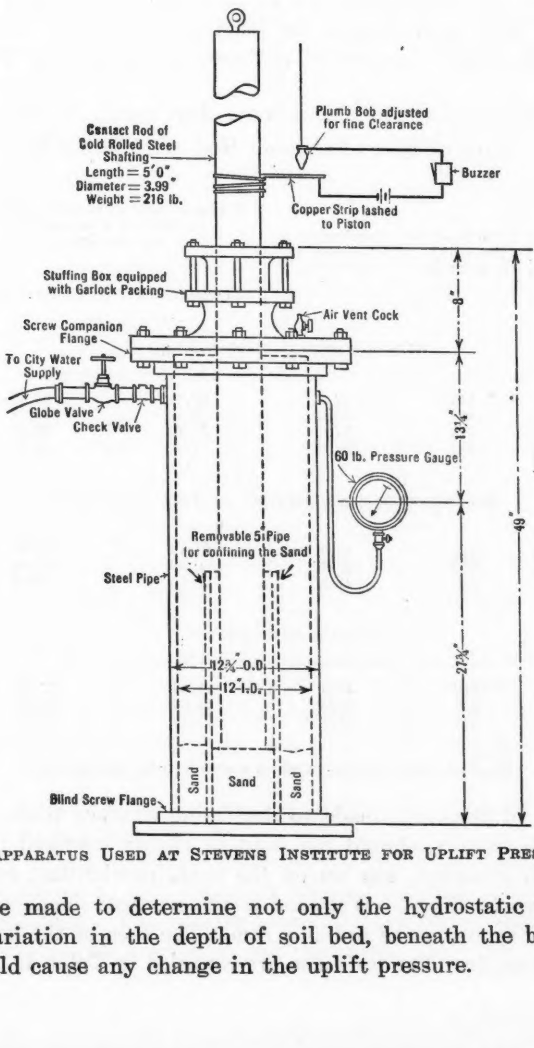

FIG. 4.—APPARATUS USED AT STEVENS INSTITUTE FOR UPLIFT PRESSURE TESTS.

Tests were made to determine not only the hydrostatic uplift, but also whether a variation in the depth of soil bed, beneath the base of the contact rod, would cause any change in the uplift pressure.

Contact Tests with Sand.—The first test was made without sand, but with the contact rod resting directly on the cast-iron bottom of the container. Although the base of the rod was machined smooth and the surface of the cast-iron bottom was rough, there was direct contact between the metals. Therefore, the effective area recorded was that portion of the area of base exposed to water. After this test, the contact rod and container head were removed. Wet sand was then put in the container to the required depth, leveled, and packed by hand. After the head was replaced, the sand was further packed by ramming with the contact rod.

The sand was brown silica building sand, having a weight (dry) of 107 lb. per cu. ft., a specific gravity of 2.5, and voids 31.2% by volume.

The tests were made by varying the depth of the sand bed below the base of the rod, from about 1 in., then to 6 in., and then to 12 in. They were conducted to find, first, the hydrostatic uplift when the rod base was in contact with the sand bed; and, second, the stuffing-box friction when the rod was just clear of the sand. The record of these tests is given in Table 4.

TABLE 4.—Results of 110 Tests with Apparatus at Stevens Institute.
(Area of Base of Contact Rod = 12.53 Sq. In.)

Sand depth below base of rod, in inches.	Total load on material, in pounds.	Stuffing-box friction, in pounds.	Hydrostatic Pressure at Base of Rod, in Pounds per Square Inch.		Average effective area, in percentage of base area.
			Friction test.	Contact test.	
Results with Sand not Confined.					
0.0*	216	44.7	20.81	23.76	87.5
0.875	216	43.6	20.72	21.97	94.2
6.5	216	37.9	20.26	20.77	97.5
12.25	216	59.2	21.96	22.27	98.6
Results with Sand Confined by Pipe Around Rod.					
1.687	216	48.9	21.14	22.19	95.2
7.125	216	40.7	20.49	21.00	97.6
12.25	216	41.6	20.56	20.68	99.4
Results with Gravel.					
1.5	216	47.9	21.06	21.55	98.0
6.5	216	54.9	21.62	21.92	98.7
13.5	216	56.6	21.76	21.57	100.0

* No sand; base of rod rested on metal bottom of container.

Another set of tests was made to determine whether there would be any difference in the results, should the sand be closely confined under the rod. A pipe, 5 in. in diameter, was set on the container bottom, concentric with the rod, as shown in Fig. 4. Wet sand to the required depth was put in this pipe and packed by using the rod as a ram. The area of the base of the contact rod is 12.53 sq. in. The tests also are recorded in Table 4.

Contact Tests with Gravel.—Gravel was put in the container and tested in a similar manner. The gravel consisted of smooth, rounded, quartz pebbles, varying in size from ⅛ to ½ in., with an occasional stone ¾ in. in longest dimension. It had a weight of 97 lb. per cu. ft., a specific gravity of 2.62, and voids 40.76% by volume. Thirty-seven tests were made. The results are recorded in Table 4.

The relation of the records are made clearer by inspection of Fig. 5, in which the results given in Table 4 are platted and curves drawn through the points. It will be noted that the hydrostatic uplift acted on practically the full area of the base when the sand depth beneath the base exceeded 17 in. Also, as the depth of the sand bed was decreased, the effective base area became less. It will be noted, also, that the confinement of the sand, by the 5-in. pipe placed around the 4-in. rod, did not materialy affect the results. With gravel, the effective area of base was larger for shallow depths of bed than with sand.

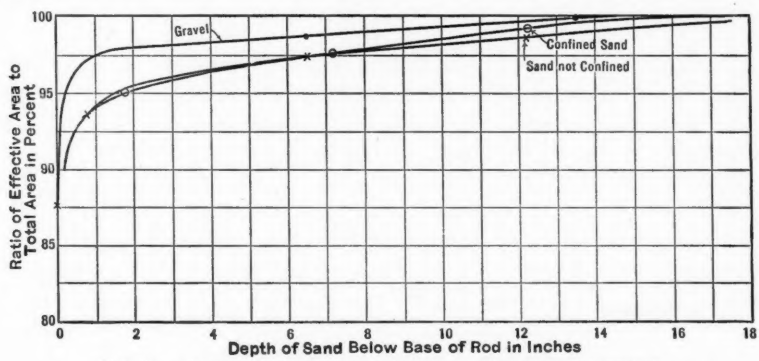

FIG. 5.—RELATIVE RESULTS OF TESTS AT STEVENS INSTITUTE.

Every test was repeated from four to seven times, and the averages of similar tests were recorded. The differences, however, in the gauge pressure readings of similar tests, were slight, as shown in Tables 6 and 7. There were 110 tests made in this series, not including many trial tests to see that the apparatus was working properly.

Contact Tests with Clay.—Clay was put in the container, and an effort made to test it in a similar manner. The clay was fairly pure in quality and of the kind used for top dressing of tennis courts.

In a damp condition, the clay firmly supported the contact rod, but it flowed when the container was filled with water and while the contact rod was used as a ram for packing. The rod sank at each ram, until its base touched the bottom of the container. An effort was made to confine the clay closely by using the 5-in. pipe around the rod, as described previously. The result was the same, namely, the clay flowed up and out of the 5-in. pipe and acted like sand with quick behavior.

When tested with hydrostatic pressure, the rod always sank into the clay bed (whether confined or not) below its original starting point as soon as the pressure was released. The results (such as they were) indicated that the effective area was nearly a 100% of the base area.

Tests Made at Rensselaer Polytechnic Institute

The apparatus consisted of a riveted steel cylinder, 36 in. in diameter by 32½ in. deep, as shown in Fig. 6. The bottom was stiffened by 2-in. angle-bars and the sides were reinforced by steel bands 3¼ in. by ⅜ in. The cover, fitted with a stuffing-box and gland, was bolted to the cylinder, and could be removed for placing sand in the container. The contact rod was a piece of 12-in. extra strong pipe, turned to 12½ in. in outside diameter and filled with concrete. A 3-in. pipe extended above the 12-in. pipe for centering the concrete weights. Three concrete weights were made, each approximately 3 by 3 ft. by 9 in. thick, with projecting steel reinforcement bars to facilitate handling. The container was tapped for a water pipe, to which was fitted a pressure gauge.

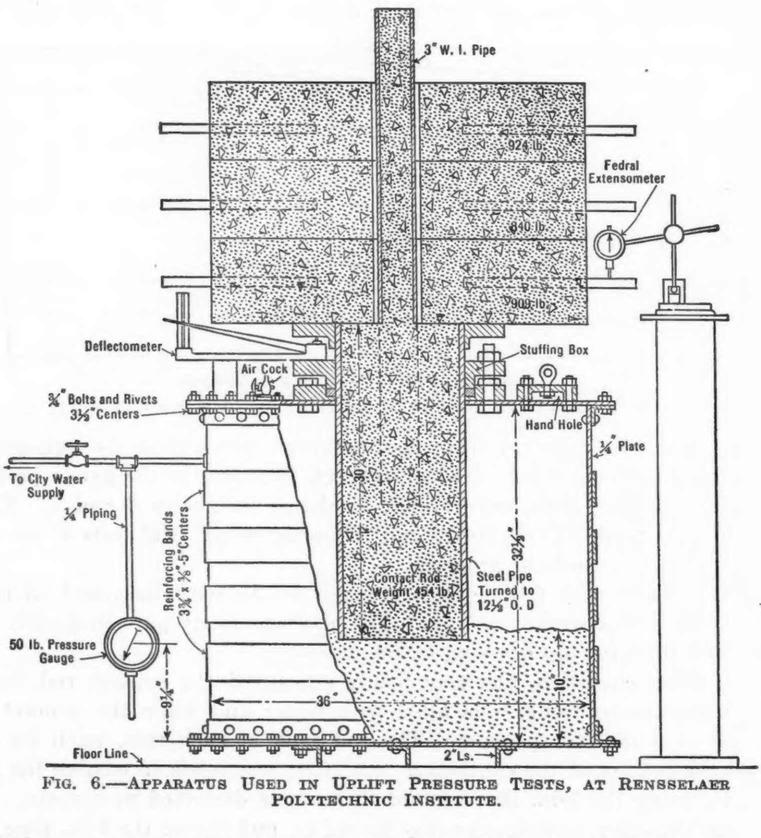

Fig. 6.—Apparatus Used in Uplift Pressure Tests, at Rensselaer Polytechnic Institute.

The contact rod weighed 454 lb. and had a base area of 122.7 sq. in. The concrete weights were 909, 840, and 924 lb., respectively. When resting on the sand for testing, the unit loadings were, therefore, 533 lb. per sq. ft. without the weights; 1 600 lb. with the lower weight; 2 585 lb. with the two lower weights; and 3 669 lb. with all three weights.

The tests were conducted in a manner similar to those made at Stevens Institute, and the mathematical formula was the same. The pressure gauge was calibrated, and the readings as corrected were recorded. An extensometer was used to indicate the first movement of the contact rod, and a deflectometer to record any change in position of the cover of the container due to the heavy internal hydrostatic pressure.

Tests were made to determine, not only the hydrostatic uplift, but also whether a variation in soil loading would cause any change in the uplift pressure.

Contact Tests with Sand.—Wet sand was put in the container to a depth of about 10 in., leveled and hand-packed. The contact rod, when loaded with weights, settled into the sand; but prior to each test, the depth of sand directly under the contact rod base was measured.

The sand was smooth or water-worn silica sand, having a weight (dry) of 116.7 lb. per cu. ft., a specific gravity of 2.83, a fineness modulus of 2.50, and voids 34% by volume.

The record of the tests is given in Table 5. It should be noted that the loading in these tests was heavy, under the three weights, being more than 1.8 tons per sq. ft. of bearing area. While the greatest care was exercised to secure accuracy, the results may seem to have unwarranted variation in the stuffing-box friction. This variation in friction was probably caused by the centers of gravity of the heavy concrete weights not being always directly in the axis of the contact rod. Any slight eccentricity would tend to bind the rod in the stuffing-box. As the recorded results are the averages of several tests (confirmed by trial tests), the percentage of base area found to be effective is no doubt close to the true figure.

TABLE 5.—Results of 28 Tests with Apparatus at Rensselaer Polytechnic Institute.

(Area of Base of Contact Rod = 122.7 Sq. In. Sand Packed Wet.)

Sand depth below base of rod, in inches.	Total load on material, in pounds.	Stuffing-box friction, in pounds.	Hydrostatic Pressure at Base of Rod, in Pounds per Square Inch.		Average effective area in percentage of base area.
			Friction test.	Contact test.	
9.25	454	262.6	5.84	5.86	99.7
10.00	1 363	353.6	13.99	14.51	96.5
9.63	2 203	319.7	20.56	20.75	99.2
9.25	3 127	619.0	30.53	30.39	100.0

The data given in Tables 1 to 5 are averages of the results found by repeating similar tests. The results of a few tests are given in more detail in Table 6, with low heads, and Table 7 with high heads.

The observations recorded in these tables show the closeness of repeated tests, which were not always made on the same day. The closeness of similar tests also shows the fairness of using averages to eliminate personal variations in manipulation and observation. Furthermore, the personnel of the

test parties was changed, and the results obtained by the various parties were in harmony.

TABLE 6.—LOW-HEAD TESTS.

$$(0.0361 \times h_0 \times a = W - w + C + F)$$

Can tested.	Depth embedded, d.	Friction curve value $\times A_0 \times d$, F.	Weight of can + contact plate, C.	Outside water head, h.	Weight of water in can, W.	Weight of water siphoned, w.	Percentage of base area effective.	Average percentage of base area effective.
WET SAND CONTACT.								
Glass jar, area = 7.07 sq. in.	1	0.214	1.27	8.06	1.6	1.22	91.0	90.5
	1	0.214	1.27	8.06	1.6	1.24	89.9	
Can, 5.53 in. in diameter, area = 24.02 sq. in.	4	2.74	8.05	21.50	18.15	12.25	89.5	88.0
	4	2.74	8.05	21.50	18.15	12.85	86.5	
Can. 9.22 in. in diameter, area = 66.77 sq. in.	6	8.0	14.82	16.50	37.95	28.30	81.6	86.7
	6	8.0	14.82	16.60	38.20	25.05	90.0	
	6	8.0	14.82	16.40	87.70	25.45	88.6	
Can, 15.09 in. in diameter, area = 178.84 sq. in.	9	23.10	22.65	20.40	129.7	55.03	91.6	94.5
	9	23.10	22.65	20.20	128.2	49.80	95.5	
	9	23.10	22.65	21.00	133.3	49.00	96.4	
DRY SAND CONTACT								
Can, 5.53 in. in diameter, area = 24.02 sq. in.	4	2.74	8.05	21.75	18.37	11.40	94.0	90.1
	4	2.74	8.05	21.50	18.16	12.30	89.5	
	4	2.74	8.05	21.50	18.16	12.80	86.6	
Can, 9.22 in in diameter, area = 66.77 sq. in.	6	8.0	15.57	16.20	37.22	32.50	72.5	74.0
	6	8.0	15.57	16.20	37.22	31.35	75.2	
Can. 15.09 in. in diameter, area = 178.84 sq. in.	9	28.10	22.65	20.25	128.80	65.50	83.5	86.2
	9	23.10	22.65	20.65	131.20	58.15	89.0	

SUMMARY

The preliminary testing disclosed that the "time element" entered the problem when dry sand was used. It required several hours for the sand to become saturated, and for the full hydrostatic pressure to be evenly transmitted. When using the large container at Rensselaer, as long as 18 to 24 hours was allowed for saturation. An experiment was made by having one end of a rubber tube embedded in the sand under the test base, and noting the time required for the water in the tube to reach the same level as that in the container.

To avoid inaccuracy caused by the time required for saturation, the sand used in the tests at both Stevens and Rensselaer was packed in a saturated condition.

By not embedding the base of the contact rod, the problem of sand friction was eliminated in the high-head tests. The experiments showed that:

1.—In pervious soils the area of base on which uplift acts is always a large percentage of the total base area. Although the results are generally between 90 and 100% of the total base area (with a few below the lower limit), it would not be safe to estimate an uplift in pervious soil of less than full pressure head acting on the total area of base, when there is no flow of water through the soil; nor less than the full pressure head indicated by the hydraulic gradient when there is flow through the soil.

The hydrostatic pressure acts, not only on that portion of the base area to which the water has access, but also on each particle of the soil material. Therefore, a part of the uplift pressure is transmitted to the base through the points of contact of the particles with one another and of the particles with the base.

TABLE 7.—High-Head Tests.

$$\left(\text{Percentage of Base Area Effective} = \frac{p}{P} \times 100\right)$$

Total load.	Material of bed.	Area of base.	Corrected Gauge Pressure for Friction = p.		Corrected Gauge Pressure for Contact = P.		Average percentage of base area effective.
			Reading.	Average.	Reading.	Average.	
216	No material........	12.53	20.81 20.81	20.81	23.87 23.86 23.56	23.76	87.5
216	Sand not confined..	12.53	20.26 20.26	20.26	20.77 20.77 20.77 20.77 20.77	20.77	97.5
216	Confined sand......	12.53	20.74 20.24 20.49	20.49	21.25 21.25 20.75 20.75 21.00	21.00	97.6
216	Gravel..............	12.53	21.26 21.26 21.51 21.01 22.26 22.01 21.76 21.51 22.01	21.62	21.27 21.27 21.52 22.52 22.52 21.77 22.27 22.02 22.27 21.77	21.92	98.7
1363	Sand not confined..	122.7	13.99 13.99 13.99	13.99	14.51 14.51 14.26 14.76	14.51	96.5

In a bed of pervious material the hydrostatic pressure acts on the portions of the surfaces of the particles which are exposed to the water. A base prevents hydrostatic pressure from acting on the top surfaces of the particles in contact with it. Consequently, the resultant of the forces on all the particles beneath the base is upward. The sum of this resultant pressure acting through the particles plus the hydrostatic pressure due to the water acting directly on the base constitutes the total uplift on the base. With sufficient depth of soil, the effective area of the base is found to approximate 100 per cent. Therefore, the portion of the total uplift acting through the particles

directly on the base is equal to the hydrostatic head at the plane of the base times that part of the base area in contact with the soil. At shallow soil depths the aggregate of the uplift forces acting through the particles fails to equal the hydrostatic head at the plane of the base times the area of the base in intimate contact with the soil.

2.—The uplift is reduced when the depth of the pervious soil is shallow. The tests at Stevens showed this reduction commencing at about 16 or 17 in. of soil. These tests also showed that the uplift is less with finer soil particles at corresponding depths of soil within the limit named. The experiments made at Stevens (Table 4 and Fig. 5) were the only ones in the series designed to record the effect of variable depth of soil beneath the base. The results of tests at Rensselaer (Table 5), in which the area of base of the rod was about ten times greater than that at Stevens, showed that the average effective area in percentage of base was approximately 98.8 when the sand depth below the base was a little more than 9 in. At a corresponding depth for the tests at Stevens (Fig. 5), the effective area was about 98.0% for sand not confined, 98.3% for confined sand, and 99.2% for gravel.

3.—The variation in sizes of base areas, from 7.07 sq. in. for the 3-in. jar to 178.84 sq. in. for the 15.09-in. can, or twenty-five times, does not alter the result.

4.—The variation in hydrostatic uplift pressure per unit of area, from 0.29 lb. per sq. in. to 30.39 lb. per sq. in., or one hundred times, does not alter the result.

The profession would be benefited if engineers would report data on magnitude of uplift pressures observed in construction work. The writer hopes that many will contribute their observations in discussion of this paper.

Acknowledgments

The writer expresses his thanks to Thomas R. Lawson, M. Am. Soc. C. E., Philip D. Joynt, Jun. Am. Soc. C. E., and Messrs. C. E. Rose and E. R. Wiseman for their help in conducting the tests at Rensselaer; to Mr. George L. Quigley for his help with all the tests and especially for his aid in revising the manuscript; to Eugene E. Halmos, M. Am. Soc. C. E., for suggestions made during the work; and to Mr. Warren R. Bentley, his Office Assistant.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

FLOOD CONTROL ON THE RIVER PO IN ITALY

By John R. Freeman,* Past-President, Am. Soc. C. E.

Synopsis

1.—This paper reports observations made by the writer on a tour in September, 1927.

2.—The Po is by far the most important river in Italy, drains one-third of its area, and flows through a broad valley rich in agriculture.

3.—The story published many times in the course of discussions on the control of the Mississippi River, that the bed of the River Po has been raised by deposit of sediment to elevations above that of the adjacent country, is found untrue.

It is true that there has been an increase in the elevation to which great floods have risen. Perhaps this is due to confining the waters within narrower limits and to the building up of deposits of sediment on the land between the main dikes and the river channel.

4.—Following the greatest flood of record, in 1926, works are now planned for raising the main dikes 3.3 ft. for many miles along the lower river.

The deposit of sediment brought down by the River Po, and precipitated where the fresh water meets salt water, has extended its delta 15 or 20 miles within the historic period, thus increasing the length of the river and adding about 160 acres of marsh land each year. A century ago the average was nearly 530 acres per year. This material comes chiefly from erosion of the slopes of the Apennines.

5.—The drainage area of the Po receives an average rainfall of 42 in. per year, varying from 120 in. on some of the elevated mountain slopes to only 20 in. in the low broad central valley, which has an average width of 40 miles for the lower 175 miles.

6.—Great floods along the Po have occurred from time immemorial and the river and its torrential tributaries have long been diked for protection.

Note.—Written discussion on this paper will be closed in **August, 1928.**

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7.—Hydraulic science was born in the valley of the Po, hundreds of years ago, and its earliest treatises relate to the control of its floods and its torrents. Leonardo da Vinci, Galileo, Torricelli, Guillimini, Frisi, and many others of the most eminent scientists of one to four centuries ago, gave great attention to its problems.

8.—Breaks in the dikes in the great flood of 1917, and the desire for increased facilities for navigation led to a remarkably complete collection of hydrographic data. The number of rain gauges in this drainage area was increased to nearly 1 000, an average of 1 to each 30 sq. miles; river discharge gauging stations to 66, including 1 or more on each more important tributary; and 123 stations were established for frequent measurement of ground-water elevation within the broad valley.

These stations provide the foundation for what perhaps is the most intensive study of a problem in river hydraulics ever undertaken anywhere in the world.

9.—Vast areas of land that are slightly below sea level, including marshes and beds of old lagoons, have been reclaimed by dikes and pumping. The cause of their being below sea level doubtless is a slow earth tilt or subsidence of the land around the Adriatic that has been observed in progress for several centuries.

10.—The main lessons from the Po which seem applicable to Mississippi and Missouri River problems, are:

(*a*) The use of double lines of dikes (or network of dikes) which presents a second line of defense when the main dike is breached.

(*b*) The maintenance of large brick storehouses at intervals of 3 to 5 miles, in which hundreds of thousands of sacks, such as grain sacks, are stored in readiness for emergencies of building sand-bag coffer-dams.

(*c*) The organization of the inhabitants so that thousands of men can be quickly mobilized in case of danger.

(*d*) A system of flood forecasts, and telegraphic warning of floods, with careful estimates of height that probably will be reached at important points all along the river.

(*e*) The prevention of "cross-over" sand-bars, at points of change of direction of curvature between the bends, by narrowing the river at these points. On the Mississippi these bars are chiefly caused by the extra width at points of contraflexure and consequent slower velocity during flood. By deposits due to this action the flood obstructs its own discharge and rises to greater heights.

(*f*) The effort to minimize cost of dredging by training the river into a single, relatively narrow, channel.

(*g*) The minimizing of expenditure for revetment where funds are scarce by trying to retain the river in the channel in which it has for a time appeared to be content, without attempting to shorten its course by short cuts across the bends.

(*h*) The present program of channel improvement on the Po is tentative and subject to revision. Some of the most eminent hydraulicians in Italy believe that a hydraulic laboratory would aid in improving methods. Plans for a research laboratory at Stra, larger than those developed for purposes of instruction at the near-by engineering schools, have been outlined; but funds have not been available for its construction.

(*i*) The work of improvement is in charge of civil (not military) engineers; with an *ex*-college professor in charge of the wonderfully elaborate preliminary hydrographic work. The whole organization reports to the Department of Public Works. Some of the most eminent engineers of Italy, consultants, construction specialists, and professors in engineering colleges, were called upon to formulate the procedure.

Introduction

Because of statements here and there in engineering literature and encyclopedias, important if true (found to be mostly untrue), that the bed of the River Po had been raised above the elevation of the adjacent lands because of its enclosure by dikes for flood control, and the consequent deposit within the river bed of sediment which otherwise would have been spread out over broad areas of flooded land, the writer has long been trying to learn more about flood conditions, dikes, sediments, and elevation of river bed along this remarkable river.

The Po (Fig. 1) is the largest river in Italy. It drains much of the southern slopes of the Alps and most of the northern slopes of the Apennines, and with its tributaries, flowing through the broad fertile plain of Lombardy—a valley filled to a nearly level floor by its sediments—has for a thousand years presented outstanding problems in river control. It was in this region that hydraulic science had its birth. It was on one of its tributaries near Milan that Leonardo da Vinci, genius of widest range in all history, practiced canal engineering and constructed the earliest chamber lock for navigation (Porte Vinciane).

It has been stated that the raising of the river bed by the increase in the deposit of sediment from year to year, has compelled the height of the dikes to be continuously increased in order to restrain the higher floods from spreading over one of the world's most fertile and intensively cultivated delta plains. Flood heights certainly have increased, but the best data available show that meanwhile the bed of the main channel has not been raised. The cause may be the gradual building up, by deposits, of the flood plain between the main dikes, and the greater confinement of the water by subsidiary dikes. So far as this condition exists, it pertains to the steeper tributaries and to the adjacent Rivers Reno and Adige, and is limited to those reaches where the steeper slope of valleys between the hills changes to the lesser slope across the floor of the main valley.

Assuming the flood level to be about the same as the altitudes of the valley cities reported in the Encyclopædia Britannica, the flood slopes would be:

Piacenza to Cremona, 62 ft. fall in 29 miles, averaging about 2.1 ft. per mile.
Cremona to Ferrara, 125 ft. fall in 110 miles, averaging about 1.1 ft. per mile.
Ferrara to the sea, 30 ft. fall in 50 miles, averaging about 0.6 ft. per mile.

Nearly all rivers decrease in declivity down stream, presenting a curved profile. Obviously, within the lower delta the slope must vary greatly with

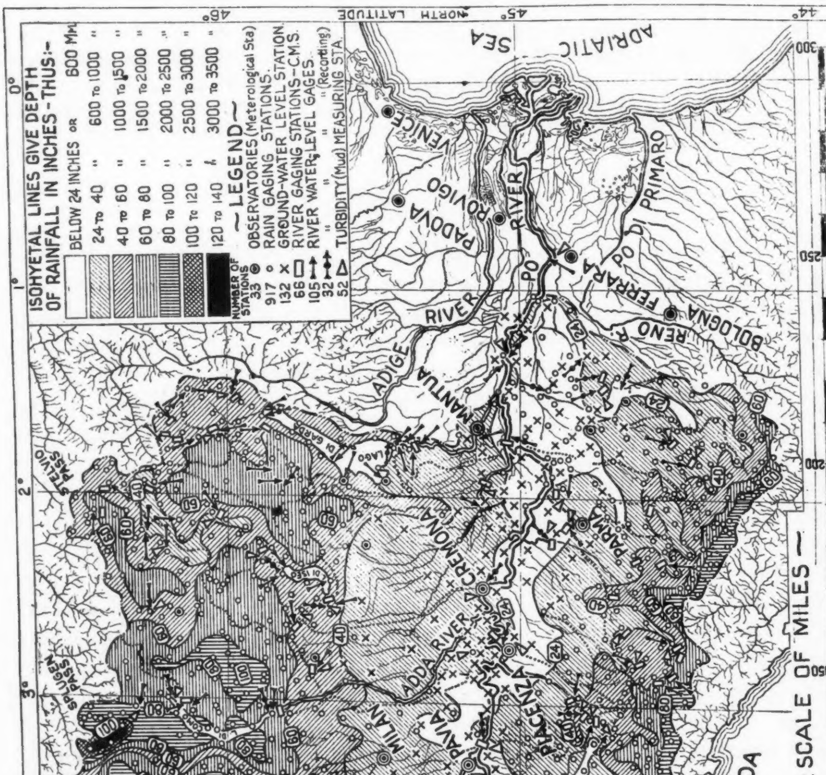

FIG. 1.—DISTRIBUTION OF RAINFALL, IN INCHES DEPTH, FOR HYDROLOGIC YEAR, 1923–24, ON THE BASIN OF THE RIVER PO, ITALY.

the river stage from flood to drought, because at 23 miles from the sea the flood surface is 20 ft. above its low-water elevation. This would give a flood slope of nearly 1 ft. per mile where the low-water slope is only about 0.10 ft. per mile.

An outstanding fact that may have had more or less effect on the down stream 200 miles of the main river, in raising both flood level and river bed, is that about 15 or 20 miles have been added to the length of the river since Roman times by the projection of its delta into the Adriatic through the deposit of sediment at its outlet, and that additional height or fall is required to carry the water over this added distance.

In the many controversies of 20 to 40 years ago, about adopting the levee system for the Mississippi, it was often stated (and also disputed) that confinement by dikes had raised the bed of the Po in Italy and of the Yellow River in China. The writer visited the Yellow River and had lines of levels run across its course at many places in the 240 miles between the crossing of the Peking-Hankow Railroad, and that of the Tientsin-Pukow Railroad, which proved those reports about the Yellow River to be mostly untrue, although not entirely so.*

Conditions Presented on the Po

The total drainage area of the Po (Fig. 1) is about 28 000 sq. miles, or nearly one-third the area of all Italy. Its breadth from north to south averages nearly 100 miles. Its length from west to east in a straight line is only about 300 miles, although about 425 miles along the curving course of the river. Nearly one-fourth this nominal drainage area, or about 7 000 sq. miles, comprising the nearly level floor of the valley, which for its lower 1 750 miles averages 40 miles in width, does not drain directly into the river because of being enclosed by dikes which while keeping out the flood water that comes from up stream, hold the rain that falls on the surface inside the dikes. Much of the rain water soaks into the porous ground, some slowly escapes through drainage channels, and a large part goes skyward in evaporation.

The average annual rainfall for the entire area is 42 in., which is about the same as on New England rivers. On some of the high mountain slopes the total annual precipitation averages more than 100 in., while it averages only 24 in. on the broad valley floor. Data for the great flood of 1927 had not been published at the time of the writer's visit.

The three successive storms that caused the previous great flood of 1917, namely, June 3 to 10, of 1.97 in. depth of rainfall, June 19, of 2.68 in., June 28 to 31, of 5.36 in. (Fig. 2), an average total of 10.01 in. of rainfall on about 20 000 sq. miles within 28 days, would cause trouble almost anywhere.

The maximum run-off of about 300 000 cu. ft. per sec. in 1917 was about 11 cu. ft. per sec. per sq. mile if reckoned on the entire area; but if reckoned on the effective area, it was about 15 cu. ft. per sec. per sq. mile. This is equivalent to a depth of run-off of only 0.6 in. per 24 hours. This seems small

* "Flood Problems in China," by John R. Freeman, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1422.

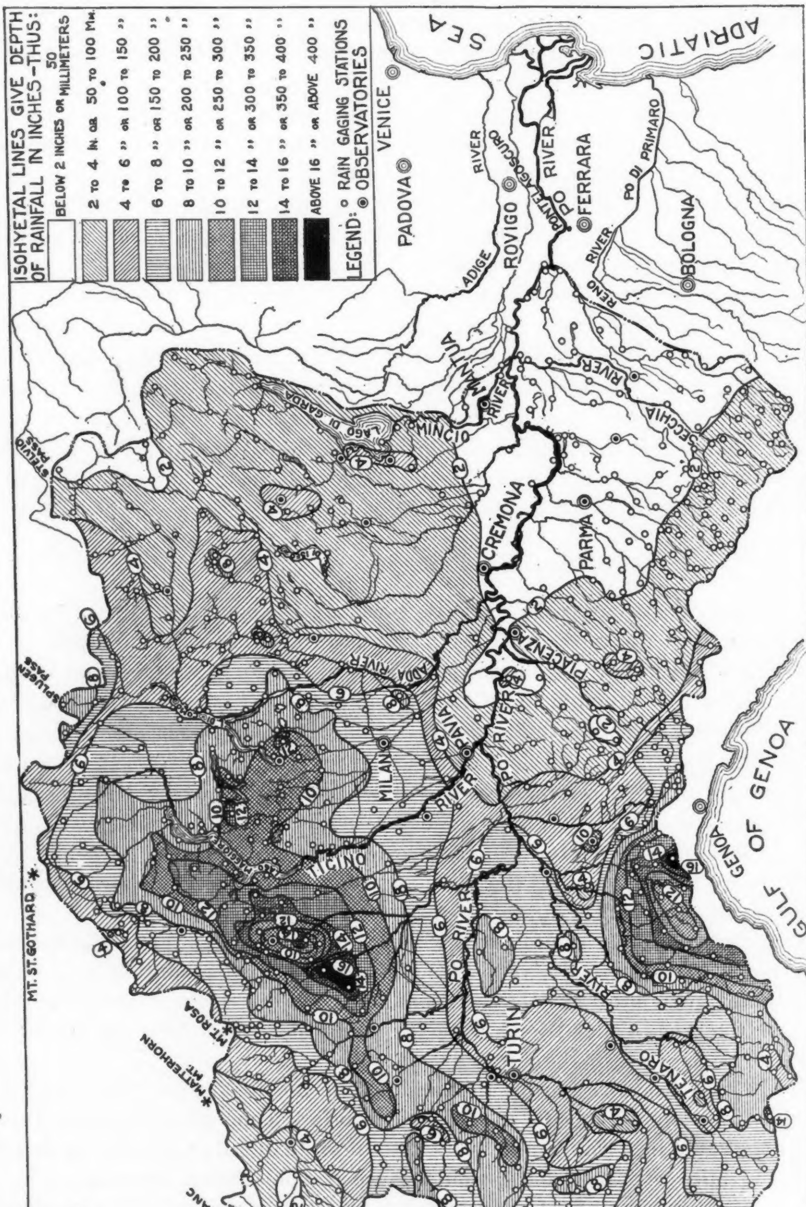

FIG. 2.—PO VALLEY, DISTRIBUTION OF RAINFALL AVERAGING 5.36 INCHES FROM MAY 28 TO 31, 1917.

for the record run-off of a century when one considers the depth of rainfall, the lay of the land, and the scant forest cover. Explanation may perhaps be found in an uncommonly absorptive soil, the surface of which at times of rain is never sealed by frost over any large proportion of the whole surface.

The northern edge of this water-shed is formed by the high Alps, rising from 1 mile to more than 2 miles above sea level; and its southern edge by the Apennines, some of the peaks of which are nearly a mile high.

The data given in Fig. 3 show a much more uniform distribution of rainfall and of run-off month by month than on most American drainage areas. The annual *Bulletin* of the Hydrographic Survey of the Po for 1923-24, published by the Department of Public Works, presents an admirably complete record, analyzed with great thoroughness. It forms a volume of two hundred and fifty, 10 by 14-in. pages, with large scale maps of the isohyetals and many diagrams of discharge.

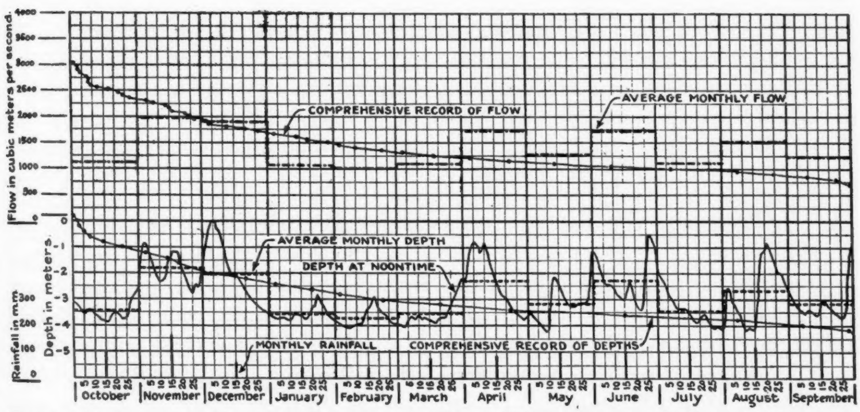

FIG. 3.—RELATION OF MONTHLY RAINFALL ON TOTAL WATER-SHED TO HEIGHT OF RIVER PO AT PONTELAGOSCURO, FOR HYDROLOGIC YEAR, 1923–24.

In brief, the deductions from the records of the 883 rain gauges in service for that year gave an average precipitation weighted according to areas adjacent to each gauge by mapping, of 1 085 mm., or 42.7 in., for 1923-24, and 982 mm., or 36.7 in., for the year next previous. On 68.5% of the drainage area it exceeded 1 200 mm., or 47 in., in 1923-24. In the six months—October to March—there was a total average precipitation of 496 mm., or 19.5 in.

The continuous gaugings of discharge at Pontelagoscuro indicate a total quantity of water equivalent to a sheet, 630 mm. (equal to 25 in.) in thickness over the entire drainage area, which, since the average precipitation was 1 085 mm., gives a run-off of 58% of the precipitation. For 1922-23, this ratio was 57%; for 1921-22, 52%; and for 1920-21, 68 per cent.

The turbidity measurements showed that during the year about 6 170 000 metric tons computed as dry silt passed the gauging station at Pontelagoscuro, which corresponds to an average erosion of 113 tons from each square kilometer of the drainage area, or equivalent to about 0.0029 in. in depth per year,

or $\frac{1}{3}$ in. per century, from the entire drainage area; but, as stated elsewhere, most of this comes from the steep slopes of the Emilian Apennines.

The lay of the land in this great valley of the River Po presents conditions on all sides which one would expect to aggravate sudden floods—high mountains, moist air swept northward from the sea, steep declivities, short swift mountain currents—all bringing the rainfall rapidly from north, south, and west, into a broad and nearly level valley floor. This floor is roughly 150 miles long by 50 miles broad, and has been formed during hundreds of thousands of years by the sand and mud eroded from the mountain slopes and brought in by the currents, filling up a broad valley that doubtless once was deep and of V-shaped cross-section, but which now presents a relatively small declivity for drainage toward the sea. Within this present broad and nearly level plain, the main river and its many tributary rivers are confined by dikes from overflowing the highly cultivated land.

That part of the delta formed within the past 2 000 years, comprising about 750 sq. miles, or nearly 3% of the entire drainage area, lies down stream from the ancient City of Ferrara and covers what once was the apex of the Gulf of Comacchio. Delta extension has been going on during the past two or three centuries at the rate of nearly $\frac{1}{2}$ sq. mile per year. Within this lower delta are vast areas of low land, part of it a few feet below sea level, that have been reclaimed from shallow lagoons and marshes by dikes and pumping, mostly within the past fifty years.

Prospects for Useful Information—Historic Backgrounds

From a study of the literature it seemed highly probable that, by a brief personal tour of observation along this river, facts could be learned that would at least be of interest in the discussion, and, perhaps, in some small degree aid in the solution of the flood problem of the Mississippi and its tributaries.

The science of hydraulics had its birth in Northern Italy and was in part developed 400 to 200 years ago by some of the greatest scientists of any age: Leonardo da Vinci, Galileo, Torricelli, Castelli, Viviani, Guglielmini, Grandi, Manfredi, Frisi, and others in their efforts to restrain these torrents, and safeguard the inhabitants along the valley and the delta of the Po.*

No region in the world was so rich in hydraulic literature up to 100 years ago, as this valley of the River Po. Some of the most ancient and famous universities and engineering schools of Europe, including those of Milan, Turin, Padua, Pavia, and Bologna, are located in and near this valley. Bologna claims the oldest university in Europe and Padua has one nearly as old. Pavia has been a center of study for more than 1 000 years. Columbus studied at its university, and, here, Volta made his first experiments. Foremost among "the old masters" of hydraulics were Michelotti, a professor at Turin; Frisi, a professor at Milan; and Poleni, at Padua. Guglielmini, who wrote the foremost treatise of his time, in 1697, on "The Nature of Rivers", was head of the Department of Mathematics in the Royal Academy of Science at Bologna and from 1700 to 1710 a professor at the University of

* The Pontine marshes, which present problems of drainage that have engaged the attention of several of the most eminent engineers of Europe, are not connected with the River Po. They are located near the west coast, about 50 miles south from Rome.

Padua.* The 11 remarkable volumes comprising, in their 5 500 pages, no less than 63 separate treatises by Italian hydraulicians of the preceding centuries, on the motion of water and the control of rivers, collected and republished more than 100 years ago under the title of the "Nuova Raccolta", was printed at Parma and Bologna within this region and related largely to the River Po and its tributaries.† Frisi's famous treatise on rivers and torrents, the world's outstanding book on these matters a century and a half ago, twice translated for the benefit of English engineers in India, deals mainly with the problems of this valley of the Po. History shows that from 1 000 to 200 years ago the Po was restless, meandering, and occasionally changed its bed. Frisi states that many centuries ago the river divided into several branches between Piacenza and Parma and converted a considerable part of Lombardy into a swamp which has since been circumscribed by dikes, while the river has been confined to a single channel. Ancient treatises presenting conditions of 900 to 1 000 years ago describe vast swamps that have since been transformed into agricultural land along the lower 170 miles of its course.

About 800 years ago the down-stream portion of the Po turned to the north, forming the Venetian branch, which gradually increased and for a time absorbed the water that formerly flowed into the southerly branches. About 325 years ago, after the Reno, which flows northward past Bologna (Fig. 2), had become so obstructed by sediment that it ceased to flow into the Grand Po, its waters spread over the shallow "valleys", or lagoons of San Martino, which thereby became filled and fertilized by sediment. Farther down stream these waters from the Reno, unconfined by dikes, without a permanent river bed, for a time ruined some of the most fertile land in Italy, formed vast swamps, and reached the sea slowly by the Po di Primaro. The Grand Po had formerly received both the rivers, Panaro and Reno, on its way to Ferrara, and, some miles below that city, it formerly divided into two branches, the Primaro and the Volano.

Frisi states further that the Reno and the Po are the two rivers that more than all others had engaged the attention of the Italian scientists, although the Arno and the Tiber also gave them much concern. The foremost mathematicians and scientists of Italy were called upon to find a remedy for the swamps and shifting waters of the Reno and the torrents lying easterly from it. They proposed to force the Reno to re-enter the Grand Po. Political obstacles, divided councils, accidents, and outbreaks long delayed the work, but finally the Reno was separated from the Grand Po and diverted into the old channel of the Primaro (Fig. 2).

Parts of the present channel of the Reno-Primaro, as outlined on modern maps, give evidence of artificial origin by their straight course. This channel was constructed within the southern edge of the great valley, and now gathers in various torrents—the Reno, Savena, Idice, Sillano, etc., flowing northward from the slopes of the Apennines, after they have dropped their loads of gravel in their separate delta cones subsequent to entering upon the more

* Bologna is in the same valley on the River Reno that discharges into the Po di Primaro, which was the main channel of the River Po in the remote past.

† A full set of these remarkable books is in the Engineering Societies Library, New York, N. Y., the gift of Professor Fantoli, head of the Engineering College of Milan.

gentle slope of the valley floor. This information and much more that can be found in Frisi's treatise and in the eleven volumes of the "Raccolta", make it plain that this remarkable river and valley have received through the past four centuries the attention of the foremost engineers and physicists of the times.

Two hundred years ago the Reno, near Bologna, formerly a tributary to the Po, seems to have given more trouble than the main river, and, in 1760, Frisi was called in for further study. He has much to say about the raising of river beds, but it pertains to rivers of much steeper slope than the main Po, and mostly applies to the Arno, which flows past Florence and Pisa. His views still make interesting reading, and indicate plainly that the instances of elevation of river bed were not on the main River Po, nor caused by confining its waters between dikes and preventing the spread of sediment over the land. The troublesome elevations of river bed occurred on the tributaries near where they left the steep slopes of the narrow valleys and entered upon the much more gentle slope across the broad valley floor, where the lessened velocity caused them to drop their burden of gravel.

Frisi argued that any new channel for carrying the waters of the Reno and the five torrents easterly therefrom to the sea, should have its pathway laid out well over into the valley floor of the Po, so as to receive the many torrents and small rivers that drain northward at a point beyond where they deposit their load of gravel brought from the hills, or beyond their gravel delta cones; and he appears to have felt certain that after a river had dropped its burden of transported gravel and carried only fine sand, it would not build up its bed. The main River Po drops its burden of gravel and coarse sand up stream from Cremona, or 160 miles above its mouth.

Frisi emphasizes gravel as a cause of restlessness in rivers, and states that the main Po drops its own load of gravel where it enters upon its relatively gentle slope at the head of the main valley, and that it receives little or no gravel from its many tributaries. Frisi's views as to the influence of gravel may have important application to the cross-over bars of the Mississippi, which are among its most troublesome features.

His books also relate that spillways for ameliorating flood conditions were actively considered along these Italian rivers more than 150 years ago, and, in general, were found to be inexpedient or unsuccessful; but these were relatively small affairs. After reading Frisi, one has a strong desire to extend his historical studies and make inspections all along the Reno from Bologna to the sea, but the writer had not time for this.

A Tour of Inspection

After preliminary talks with two or three of Italy's foremost hydraulic engineers, Professor Gaudenzio Fantoli, at Milan; Luigi Luiggi, Hon. M. Am. Soc. C. E., and Mr. Lorenzo Allievi, at Rome, who provided kindly introductions to the chief engineers of river control in the Department of Public Works, the writer recently spent four or five busy days inspecting the delta, the dikes, and the channel of the Po and the Adige Rivers. He traveled by automobile back and forth about 200 miles, largely on well-kept roads on tops

of dikes, crossing and re-crossing the rivers repeatedly by bridge and ferry, and thus seeing much of them, their dikes, and the valley floor. This inspection included much of the way from Piacenza, past Cremona, Parma, Mantua, Padua, Ferrara, and Rovigo to the Adriatic, thence back to Ferrara, and across the valley floor to Bologna. The centuries of intensive cultivation and sub-division have given far more numerous cross-roads than along the Missouri, and nearly every road traveled was on top of an embankment that would form a secondary dike in case of need.

Many of the main dikes were said to be hundreds of years old, but had been repaired, raised, and thickened from time to time. Their recent permanence of location seems to indicate a less unruly river than the Missouri or Mississippi. Nevertheless, one look at the map of roads embanked on top of secondary dikes and at their curves which evidently follow old river bends, shows that this river has in past years sometimes meandered far outside its present pathway. (See Figs. 4 and 5.)

On most of the trip the writer was accompanied by one or another of the engineers in charge of the new work. The journey was extended by boat to the end of the principal delta mouth, where works are in progress for creating and maintaining a broad channel about 1 000 ft. wide, through the bar, suitable for barges of 600 tons capacity.

The tour was full of interest, the roads were excellent, and the farming was intensive, the fertile ground being burdened with crops, rice, maize, hemp, and forage. Economy in use of the land was shown by the long rows of trees along the cross-roads, planted to give forage for silk worms, and by occasional recent plantings on the foreshore, outside the dikes, of long parallel lines of poplars, about 12 ft. apart, for future wood pulp. There are now many broad strips of land of small utility lying between the main dikes and the river, which apparently it is the intention to reclaim for more intensive cultivation after the flood channel has been further stabilized and narrowed by works now in progress.

Everywhere the greatest courtesy was extended, many maps and reports showing features of special interest were furnished, and one was made to feel that the Italian engineer has a most kindly feeling toward his American coworker. Space permits only a few of the observed features of special interest to be given. It is hoped to prevail on one of the engineers prominently connected with these works to prepare an article for translation and publication in some American engineering periodical, which may state more adequately the remarkable story of the works now in progress.

What Was Seen and Heard on the Tour

All things considered, the writer knows of no more ambitious, courageous, and carefully planned project of river regulation now going on anywhere in the world than that in progress upon the Po, nor of any which rests on a more painstaking preliminary scientific study of the river itself.

For more than five years, a new hydrographic survey has been going on in the Valley of the Po, largely under the direction of Professor Mario Giandotti, following the recommendations of a Royal Commission of Engineers

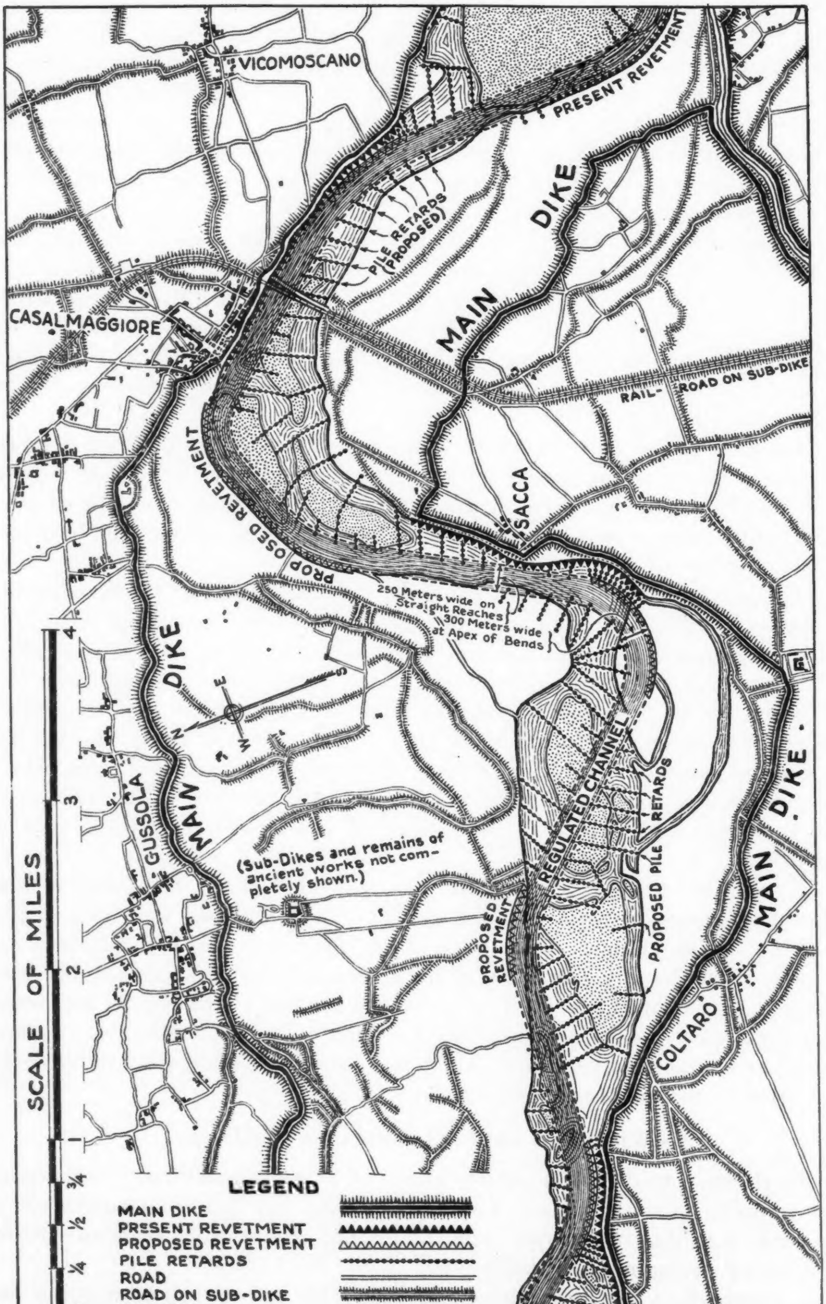

FIG. 4.—PRESENT AND PROPOSED CHANNEL AND REGULATING WORKS NEAR CASALMAGGIORE IN 1919.

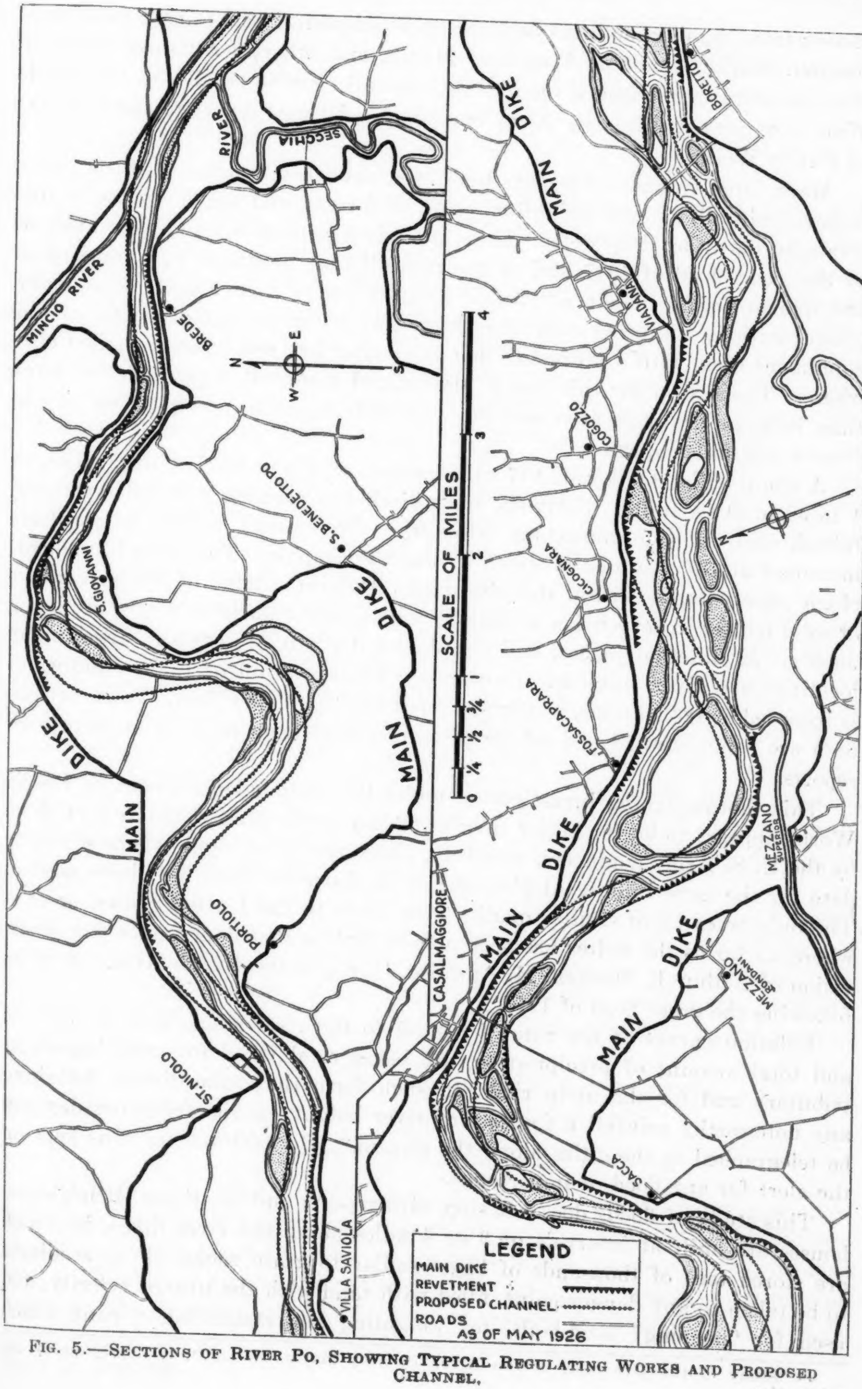

FIG. 5.—SECTIONS OF RIVER PO, SHOWING TYPICAL REGULATING WORKS AND PROPOSED CHANNEL.

chosen from the most eminent in Italy, on which were Luigi Luiggi, Gaudenzio Fantoli, Mario Giandotti, Francesco Mauro, and others. Although many of the engineers now engaged on the work were in active service in the World War, they are not primarily Army engineers. All now report to the Ministry of Public Works.

Many large volumes of reports have been published relative to storm paths, isobars, rainfall, run-off, irrigation, effect of forests, and the character of sub-strata as shown by deep wells. These include a remarkably complete analysis of the great flood of 1917 and of the rainfall that produced it, comprised in 186 quarto pages. Their admirable isohyetal maps (see Fig. 1) show rain-gauge stations scattered over the thirty-two tributary sub-divisions of the catchment area, with a profusion that the writer has never seen equalled elsewhere. The reason for this lies in the rugged mountain topography of more than half the drainage area and the wide differences in precipitation at stations a few miles apart.

A count (Fig. 1) reveals 917 rain gauges, within this 27 500 sq. miles, or 1 to each 30 sq. miles. Sixty-six river discharge gauging stations are maintained, one on each important tributary. The river's height is regularly measured at 137 stations, of which 32 are autographic. There are 33 meteorologic observatories within this drainage. Determinations of the percentage of solid matter in suspension at various stages, particularly in floods, are being made at 52 different places, and samples of deposits of sediment from many localities are given mechanical analysis. The elevation of the ground-water is recorded at 123 stations, widely scattered over the valley floor. These various data are being studied and tabulated in excellent form in a series of printed reports.

This thoroughgoing investigation under the Italian Department of Public Works appears to be far better than anything of the kind that has been done by the U. S. Army engineers, or others, along the Mississippi, where scientific data for the more intelligent planning of flood control have long been needed. The only research of similar quality ever made in the United States, or elsewhere, as far as the writer has been able to find, is that done under the supervision of Arthur E. Morgan, M. Am. Soc. C. E., on the Miami River, in Ohio, following the great flood of 1913.

Relation curves of the rate of run-off to the river stage, also to the rate and total amount of precipitation, are being established for each important tributary and for the main river, in such form that immediately following any noteworthy rainfall a forecast of river height for the following day can be telegraphed to the cities along the stream.* Supervisors are thus kept on the alert for any flood of importance.

This study of floods does not stop with reports and treatises. Brick storehouses are built at intervals of 3 or 4 miles along the river dikes, in which are stored tens of thousands of bags similar to grain sacks, all in readiness to be taken to any danger-point, filled with sand with the utmost celerity, and used for "hooping" sand-boils, for weighting and stabilizing a bank which

* "Study of the Dynamics of the River Po," by Mario Giandotti, is nearly ready for publication.

threatens to slip because of saturation or pressure, or for building sand-bag coffer-dams on top of the dikes. The peasant farmers of the broad countryside are organized in readiness for extreme floods, each man knowing his appointed place along the dikes, so that literally thousands of men can be mobilized at a few hours' notice for defence at any point.

Proof of the efficiency of this organized foresight was given in the flood of 1926, which was the greatest ever recorded in the history of the Lower Po. In May, 1926, following extremely heavy and widespread rainfall, the river rose to a height averaging about 1.3 ft. higher than the top of the main dikes for many miles along the Lower Po, but was prevented from overflowing them. About 30 000 men are said to have been mobilized and, in addition to other work, built a continuous line of sand-bag coffer-dams, from 2 to 3 ft. high, and more than 30 miles in total length, so promptly and quickly that there was no break in the dikes down stream from Casalmaggiore, or along the lower 130 miles of the river. This was a wonderful performance, considering that the period of warning was remarkably short, and that the tributaries entering the flat valley floor are mostly short, steep, mountain torrents. The distance in an air line from the farthest corner of the water-shed up stream from Casalmaggiore is only about 190 miles.

The middle of the Po Valley, above Casalmaggiore, did not fare so well in the flood of May, 1926, or in that of June, 1917. Here and there were breaks and overtopping of dikes such that a large area of farm land was put under water, and city streets were flooded 2 to 5 ft. in depth. Photographs of the flooded districts bear striking resemblance to scenes the writer has witnessed along the Mississippi. There were the same sand-bag dams on top of dikes, the same weighting of insecure dike slopes with sand-bags, the same hooping of sand-boils, and a similar gathering of women, children, and household goods on the tops of the dikes that protruded from the flood. It is a strange coincidence that the 1926 flood on the Po and the 1927 flood on the Mississippi should have each attained the greatest height ever recorded.

Within the upper part of the delta plain of the Po, up stream from Casalmaggiore, more fortunately than along the Mississippi, the vast network of interior dikes, one of which forms the base of almost every road and cross-road, appears to have restrained the flood that penetrated the main dikes. Judging from their shape, some of these old dikes mark the outline of river beds of centuries ago.

Works are now in progress for many miles along the Po, for raising the main dikes 1 m., or to 3.3 ft., above their present elevation.

Distribution of Rainfall

The two maps, Figs. 1 and 2, are selected from among many contained in recent reports, as showing the remarkably uneven distribution of the precipitation caused by the contrasts in elevation and exposure to vapor-laden winds. The first (Fig. 1) shows the distribution of the average total precipitation for the entire year; and Fig. 2 shows that for the $2\frac{1}{2}$ days, May 28 to 31, which caused the great flood of 1917, one of the most severe recorded within the past century.

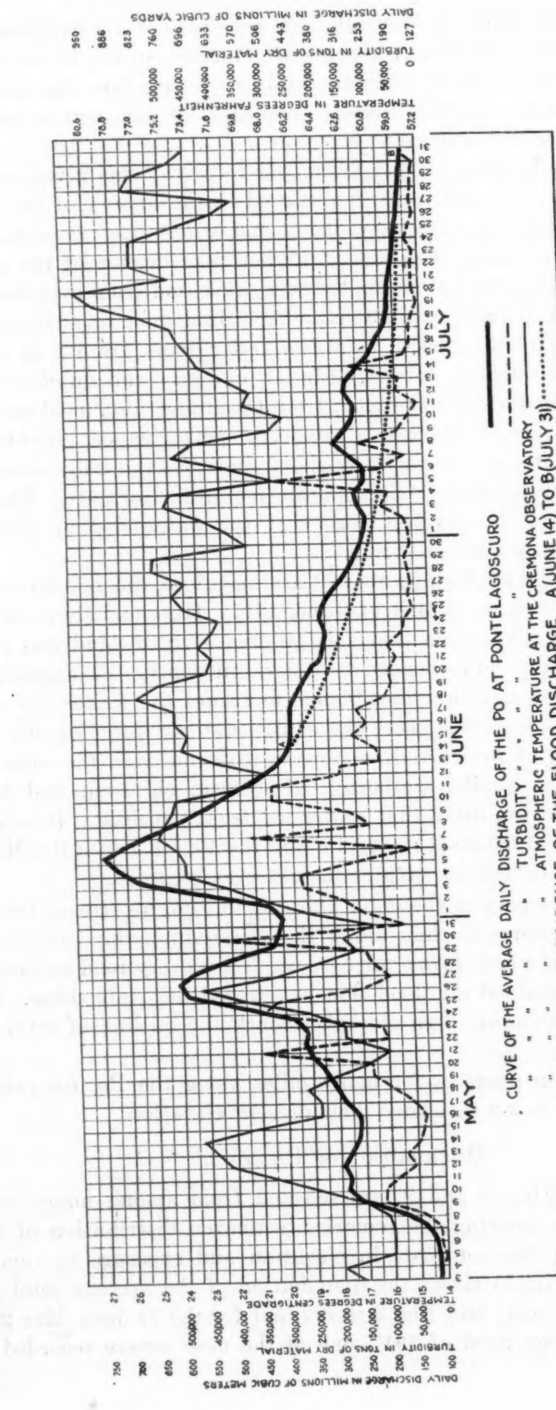

FIG. 6.—CHARACTERISTICS OF THE RIVER PO DURING FLOOD OF MAY-JUNE, 1917.

In Fig. 6 is shown the effect of the rapid succession of three heavy rainfalls which saturated the ground and led to the flood peak of June 1 to 5. It furnishes a measure of the flood in terms of cubic feet per second, while Fig. 7 presents it in terms of elevation of the river surface above the ordinary or low-water height at several important cities. Fig. 3 shows the daily rate of discharge for the hydraulic year, 1923-24, as well as the total number of days in the year upon which the river was above or below a given height or volume.

A glance at Figs. 1 and 2 shows the wonderful profusion of rain-gauges, stations for ground-water observations, river-gauging stations, also the wide variation of precipitation. The yearly range is from more than 120 in. in depth on mountain slopes to less than 20 in. in parts of the broad low valley, a ratio of 6 to 1.

From an inspection of Figs. 1, 2, 3, 6, and 7 it is plain that uncommonly complex problems of hydrology are presented on the River Po. The reports mentioned previously contain many other diagrams of scientific and practical interest. A map showing in detail the successive positions of this river within the historic period of 2 000 years would be of much interest, notwithstanding that for a century or more this stream has been held to the location in which it is now found. The writer knows of no such map, but the story of the control of this river will not be complete without one.

The Dike System

A different arrangement of the dikes is presented along the lower river, below the ancient City of Ferrara, from that along the Upper Po. Along the lower river reliance appeared to be placed wholly on one main dike on each side of the river, whereas along the upper river there was a network of dikes. Along the Adige River near Rovigo there was a single line of dikes on each side, which appeared to be about 25 ft. high. The proximity of many ancient appearing buildings to these massive dikes indicated they had long held this river to its prescribed course.

At nearly all points seen by the writer, the main dikes of the Po had a top width of at least 16 ft., in other places of 23 ft., surmounted by a well graveled highway, excellently maintained. This appears to be better practice than that which prevails along many of the Mississippi levees, where the roads are kept off the top and placed along the inner terrace or berm, for the reason that water collecting in wheel ruts weakens the levee. Plainly, the remedy is to maintain a good hard-surfaced road width without ruts and with a high crown. Travel along the top of a dike scares away muskrats from burrowing into it.

The slopes on the river side were 1 vertical on 1½ horizontal. At the concave bends this river slope was roughly paved above low water, and was revetted below low water with a loose pile of stone rip-rap a few feet thick; so that in case of undercutting, stones could slide down and protect the base. On the land side, the slopes were 1 vertical on 2 horizontal, with one or more berms 5 to 10 ft. wide, at intervals of about 15 ft. in elevation. The height of the Po dikes above the fields was apparently about 25 ft., at a distance of 125 miles

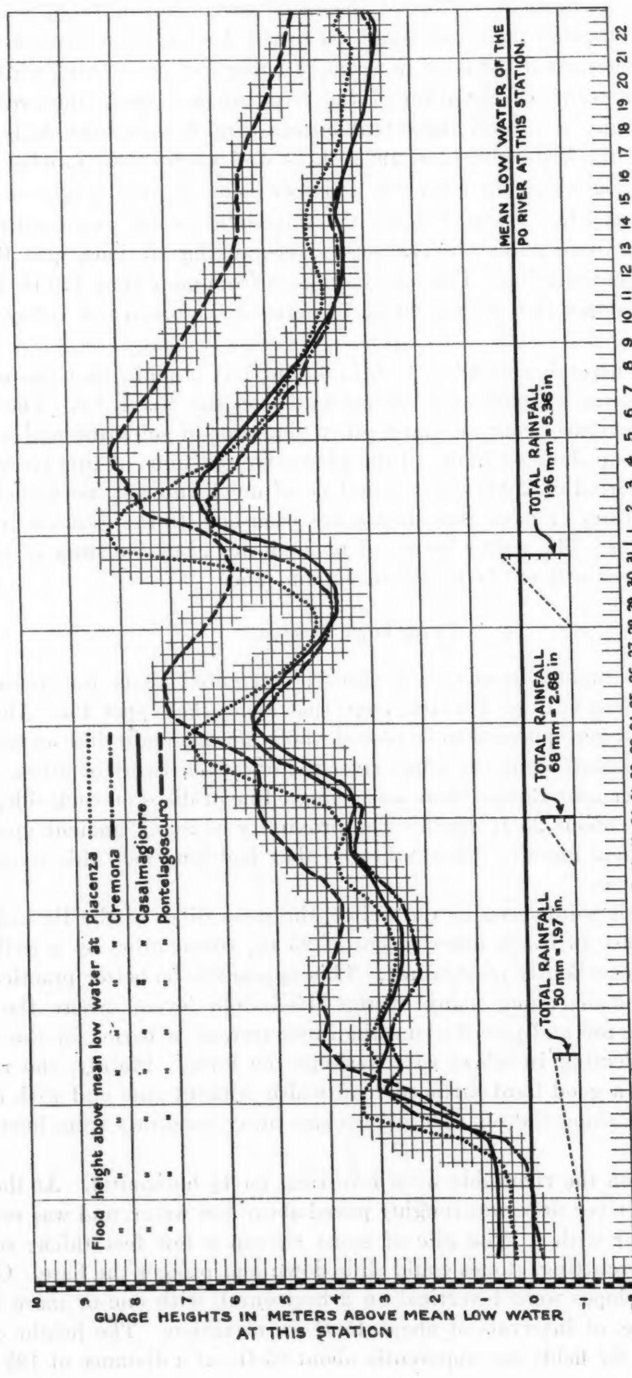

FIG. 7.—FLOOD HEIGHTS AT PIACENZA-CREMONA-CASALMAGGIORE-PONTELAGOSCURO DURING THE PO RIVER FLOOD OF MAY-JUNE, 1917.

from the sea. At some places the height is said to be greater and dikes 45 ft. high are said to exist on some of the tributaries.

Dike at Concave Bends.—Fig. 8 shows a standard cross-section of a sturdy structure, 23 ft. wide on top, well adapted for a highway, and considering the quality of fine sand of which it is built, sufficient in thickness to escape saturation and sliding during the short-lived floods of the Po. The sand of which these dikes are built appears in general to be composed of grains $\frac{1}{50}$ to $\frac{1}{100}$ in. in diameter, through which saturation would proceed slowly; but that they are sometimes rendered dangerous by saturation was indicated by photographs taken during the great flood of 1926 of inshore banks completely covered with a layer of sand bags to stabilize them. This condition, however, was the result of three successive floods within 28 days, illustrated on Figs. 6 and 7.

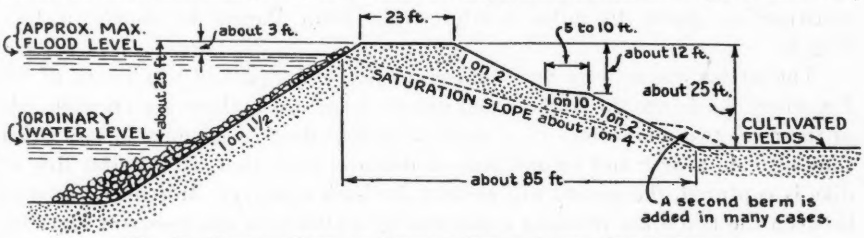

FIG. 8.—TYPICAL DIKE AT CONCAVE BENDS ALONG THE PO RIVER.

The material for dike building so far as noted, was extremely fine sand like that in the river bed and seemed to present no large proportion of slippery colloidal material. The conditions for dike building, as to material and substrata, appeared more favorable than along the Mississippi, with fewer soft swamps or old bayous to be crossed by the dike. Whatever of these soft spots there may have been in the days of earliest dike building, had been obliterated by centuries of intensive farming. Also, the shorter duration of the floods on the Po than on the Mississippi presents smaller chance for percolating water to penetrate through the dike and thereby softening and weakening it.

The only place where sub-surface conditions could be observed was at excavations for a new navigation lock on the river bank at Ferrara, for navigation from the main River Po southward through a canal to the Po di Primaro and the River Reno. Here, the ground for a depth of about 10 ft. below the natural surface was of clean, fine sand that apparently would pass a sieve of 50 meshes to the inch and was tolerably free from organic matter. Beneath this, the excavation showed a darker colored stratum of rather stiff, clay-like material of a consistency so weak that the steel moulds for concrete sheet-piling, apparently 30 or 40 ft. in length, sank rapidly beneath the dead weight of a heavy pile-driver ram, without a blow, to a depth of 20 ft. and were sent to full depth with relatively few 5-ft. drops of the ram.

From all that could be learned in this brief visit the dikes along the Po and Adige Rivers were in far less danger from undercutting and sliding inward during a flood than those along the Mississippi. This might be expected, because of their smaller depth which, in greatest flood, presents a bank-face on the concave shore of only one-half or one-third the height found in many places along the Mississippi. Moreover, the writer judged that along

the Po there is an absence of submerged snags or water-logged trees from caving banks to give the current a vicious twist against the bank. Local conditions apparently do not call for a broad protecting mattress on the river bed at the foot of the bank slope as on the Mississippi. Safe resistance to the underscour of the flood currents of the Po, even at the sharpest concave bank, apparently can be secured by a relatively thin breastwork of fascines, pinned down and stiffened by a few rather slender piles, and weighted with stone rip-rap.

A completed training wall of this kind near Casalmaggiore, which successfully withstood the great flood of 1926, was inspected. Another training wall, now under construction, about 10 miles up stream from Parma, also was inspected. A cross-section of the dike combined with a training wall now under construction about 10 miles northwesterly from Parma is shown in (*E*), Fig. 9.

The writer was told by Senator Luiggi that for a part of the length of the Po, where floods are the highest and danger is greatest, there are on each side of the river two main dikes in parallel, at a considerable distance apart, which thus provide a first and second line of defense, such that, if the first line of dike is ruptured, the second will protect the back country. Moreover, the area between the two dikes presents a noteworthy addition to the reservoir capacity of the river. The writer did not happen to see any clear cut examples of this double dike with reservoir space between, but found in the region between Cremona and Casalmaggiore an equivalent protection afforded in the multiplicity of secondary dikes beneath the highways and cross-roads, some of which on curved lines parallel to the river, apparently marked the shore line of centuries ago. (See Fig. 4.) In many places a third lesser dike has been built close to the water's edge by the farmer to permit the utilization of an additional strip of ground, with the expectation that this will be drowned out once every 5 or 10 years.

There is, moreover, a system of major and minor dikes along much of the river, as illustrated by Fig. 10.

Everywhere, at short intervals along all the dikes, were broad ramps carrying the road down to the level of the cultivated fields. Some of these main outer dikes present a slightly zig-zag alignment, for which there was said to be some military reason, there being a main highway along the top.

This network of main roads and cross-roads, each on top of an embankment about as high as the main dikes, which were found all along the river from Cremona to Parma, presented a great safeguard against widespread inundation, which was mostly absent down stream from Ferrara to Rovigo, or within the lower delta. Districts along the down-stream portion of the river obviously need less substantial safeguards than midway of the stream, because of their multiplicity of old channels by which the water can escape quickly to the sea.

The River Channel

The average impression left in the writer's memory, after inspecting the River Po at many points from Piacenza to the sea, at a stage about 3 ft. above

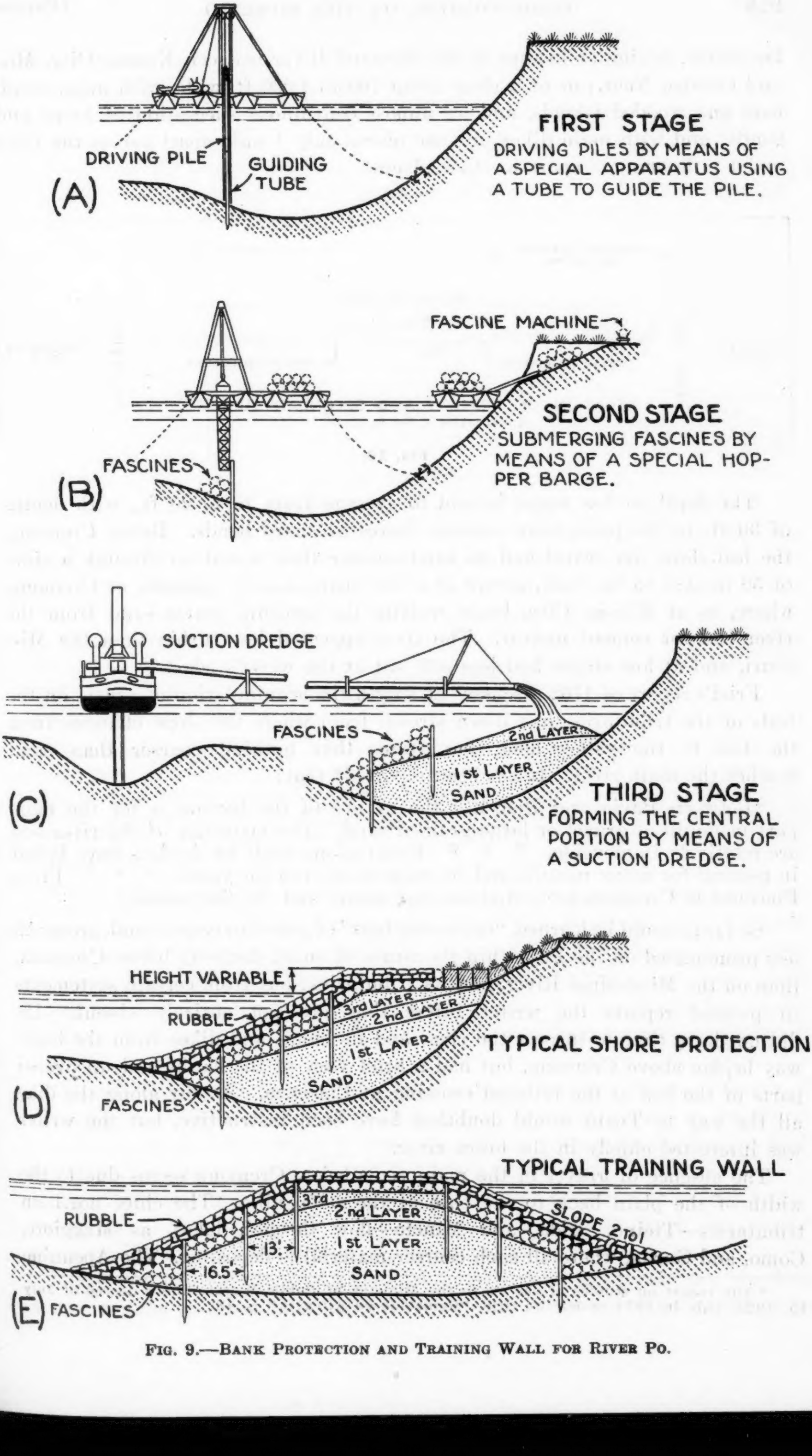

FIG. 9.—BANK PROTECTION AND TRAINING WALL FOR RIVER PO.

low water, is similar to that of the Missouri River between Kansas City, Mo., and Omaha, Nebr.; or of a river about 700 to 1 000 ft. wide, with many sandbars and wooded islands, and an almost continuous succession of loops and bends; and with main dikes in some places only ¼ mile apart across the river and 1 mile or more apart in other places.

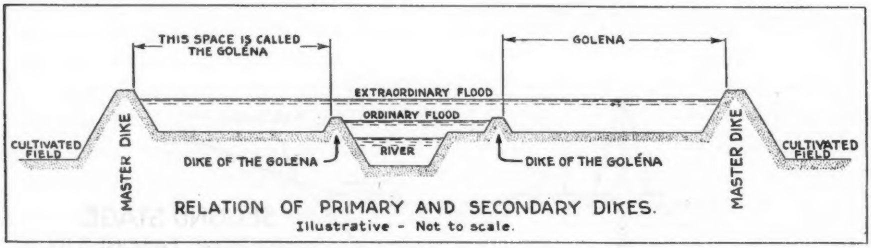

FIG. 10.

The depth at low water is said to average from 10 to 20 ft., with depths of 30 ft. in the pools near concave shores or sharp bends. Below Cremona, the bed shows no gravel and no sand coarser than would go through a sieve of 50 meshes to the inch, except at a few spots, as, for example, at Cremona, where, as at Kansas City, boats reclaim the medium coarse sand from the river bed for cement mortar. The river appeared less turbid than the Missouri, and at low stages had less soft silt at the water's edge.

Frisi's report of 1762 discusses at length the accumulation of gravel on the beds of the tributaries just down stream from where the slope changes from the hill to the valley floor, but states that nothing coarser than sand reaches the main river. It has been reported* that:

"Between Pavia and Piacenza the nature of the bottom is for the most part made up of gravel or large grained sand. The materials of the river bed are relatively immovable. * * * Excavations made by dredges have lasted in general for many months and in some cases even for years. * * * From Piacenza to Cremona are found the last gravel and the first sands."

So far as could be learned, "cross-over bars" of gravel or coarse sand, are much less pronounced on the Po within its course of small declivity below Cremona, than on the Mississippi River, and its tributaries; but from certain statements in printed reports the writer infers they are not entirely absent. He did not have time to inspect the river and its banks and dikes from the highway bridge above Cremona, but had a brief view of the banks and uncovered parts of the bed at the railroad crossing at Piacenza. A tour along the dike all the way to Turin would doubtless have been instructive, but the writer was interested chiefly in the lower river.

The absence of gravel in the main river below Cremona seems due to the width of the plain back to the base of the steep hills. The chief northern tributaries—Ticino, Adda, and Mincio—flow through lakes, as Maggiore, Como, and Garda, where all such coarser material is deposited. The Apennine

* The report on studies of the Po to the Ministry of Public Works, under decree of July 15, 1922, pub. in 1924 under the title, "Relazione ed Allegati," p. 48.

tributaries have no such lakes. Nevertheless, apparently, they fail to deliver gravel across the great width of the valley floor to the main river.

The proportion of fine silt and of slippery colloidal material in the main river when only 3 ft. above its lowest stage looked rather small, and perhaps this may simplify the problems of safe dikes with rather steep slopes.

These are the impressions left upon a rapid traveler. To answer all questions about bars, bends, caving banks, and stability of channel decisively would take far more time and facilities than were available.

Regarding Navigation

Increase in navigability is one of the chief purposes of the extensive works for river training, recently begun after several years of study, as previously described. Sand-bars in a few places in the lower river and gravel bars in the river above Cremona have presented obstacles on which considerable work by suction dredges has been performed in the past. It is now hoped that, by training the river into a single relatively narrow channel of nearly uniform width, the requirement for dredging will be largely decreased. The report to the Department of Public Works—previously quoted—expresses the hope (pages 48 to 50) that the delivery of sedimentary material into the river will at some future time be lessened

"* * * by regulation works on the mountain sides, which, while adding to the safety of the habitations, and that of the roads and other public works, will also greatly help in the better maintenance of the depth for navigation by rendering the water less turbid."

Also, this report states:

"The dredging work may in time be notably reduced where, with the appropriate work of retaining, reforestation, and draining, provision is taken to regulate the mountain basins of the Apennine torrents, works which will undoubtedly result also in the efficient consolidation of the landslides which, especially in the Emilian Apennines, continually menace the roads and habitations."

Fifteen miles up stream from Casalmaggiore, the writer saw 600-ton barges laden with rip-rap from the hill quarries north of Padua, which he was told had navigated the river up stream from the canal lock about seven miles easterly from Adria without difficulty. No other boats of importance were noted except some fishermen's boats near the mouth. Evidently, navigation of the Po has not yet become active. Perhaps it awaits improvement at the lower mouth.

At the delta mouth there is a sudden shoaling from the ordinary depth of 10 to 20 ft. to from 2 to 5 ft. at low water for about ½ mile in length of course. From this bar, under present conditions, navigation for barges with drafts of 7 or 8 ft. may now be readily accomplished.

In the writer's opinion this shoaling at the exit doubtless is caused by the precipitant action of the sea water on the fine sediments which are carried along readily in fresh water by moderate currents.

Frisi and other writers, of from 100 to 200 years ago, mentioned shoaling at the mouths of other chief Italian rivers, and presented various theories as

to its cause. The favorite explanation had to do with drift, under the action of littoral sea currents, of sediment brought down by the river.

River Training in Progress on the Po

As a part of the recent widespread industrial activity in Northern Italy, river training has been begun for developing navigation on the Po and its tributaries. Connection by locks is to be made with the canals connecting with the Adige and the Reno to ports on the Gulf of Venice and on the Adriatic Sea, thereby serving the river cities all the way up to the great industrial districts of Milan. These works are designed to provide for steel barges of 600 to 800 tons capacity and of 8 ft. draft. The plans comprise, chiefly, training of the river into a curving channel of nearly uniform width of about 1 000 ft., with a narrowing at points of contraflexure instead of an expansion, such as is found in many places along the Mississippi, and which extra width, as explained by the late James B. Eads, M. Am. Soc. C. E., may be largely responsible for its bad cross-over bars.

The regulation of the Po is designed to give ultimately a width of 300 m. for the bends and only 250 m. for the transition sections between the bends.

This great public work was actively begun about 1926 after years of patient study and planning under Professor Giandotti and others. It is understood that the present plans are tentative and subject to modification as the work proceeds. In brief, the system is designed to make a limited amount of money go as far as possible with safety. It is proposed to protect and hold most of the concave shores in their present position by a revetment composed of fascines and rip-rap as outlined in (*D*), Fig. 9, without any mattress work. Where the river has to be narrowed at a wooded island or a "middle ground", training walls will be built of a type similar to that shown in (*E*), Fig. 9. The lesser depth permits these structures to be of much simpler character than has been found to be permanently reliable on the Mississippi.

It is hoped by those in charge that the total length of protection, by means of fascine and rip-rap like (*D*), Fig. 9, which will ultimately be required, will probably not exceed one-fourth the entire length of the river bank. This limited application of bank protection is obviously a matter of great economy, and it is expected that by holding the river thus narrowed to a uniform width, somewhat rigidly to its present curving course, in which it has long seemed content, and retaining its present length, elevation, and slope, that the Po will be satisfied to remain within the path now laid out for it, with protection only along the concave shore. The writer understands that the uniformity of width, with shore lines at the bends curved somewhat precisely to prescribed radii, all as shown on certain of the plans portions of which are copied on Fig. 5, is not expected to be produced forthwith, but is the ideal shore line toward which all the revetment and training walls will tend.

This retention of the present total curvature leaves the river 50% longer from the Adda confluence to the confluence with the Secchio, 82 miles down stream, than if the bends were cut out; in which case doubtless both banks would have to be reveted most of the way. Here and there, where the river is showing a tendency to dig too deeply into a concave bank and thus threaten

the adjacent dike, one or more spurs of fascines and rip-rap have been built out at a slight angle to the current, to turn it back toward mid-stream. The success with which the dikes, remarkably close to the edge of the low water, are maintained by light rip-rap, and by protecting only the concave bank, indicates that the Po is far less restless within its banks than the Mississippi, and that its subsoil presents no such treacherous conditions as those which without warning caused the crevasses at Poydras and Wecama, La., and threats of crevasses at Tunica, Miss., Stanton, Miss., etc., during the Mississippi flood of 1922. There may have been difficulties of which the writer did not learn. The few days spent along this river were far too brief for studying all its possible vagaries.

It was of special interest that engineers of great experience and manifest ability, like Giandotti, Fantoli, Luiggi, Mauro, and others of eminence, had evidently concluded that they could safely devote the limited funds available for improvements for navigation and flood control on the 82 miles of the River Po between the Adda and the Secchia, to confining it to a uniform width of about 825 to 1 000 ft., following nearly its present channel, without important change of total curvature, without making cut-offs across bends for straightening, and with a total length of special bank protection and revetment amounting to only about 25% of the bank. The projected channel section is shown in Fig. 11.

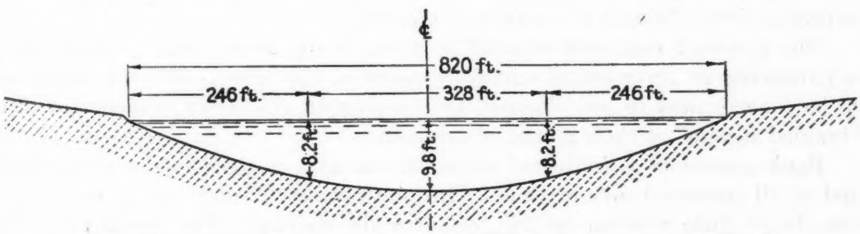

FIG. 11.—PROPOSED NORMAL SECTION OF REGULATED CHANNEL FOR THE PO.

The writer would have favored a more radical straightening and cutting out of bends, beginning near the mouth, thereby lowering the flood heights on the upper river and shortening the path for navigation, all, of course, subject to study and experiment in the field and in a hydraulic laboratory. Such a radical innovation would have cost more for the early stages and would have delayed the opening of systematic barge navigation. The method adopted is in the nature of "playing safe" and minimizing the immediate outlay.

In some reaches within this distance the space between the main dikes is now about a mile in width. Within this space the river has been free to meander, and alongside its main channel there have been deposited banks of sediment to such a height that the shallow depth over them in a great flood adds relatively little to the discharge capacity. The controlled narrowing to a uniform width of 825 to 1 000 ft. will add largely to land available either for cultivation of narrow forests of quick growing pulp wood, or for field crops enclosed by cheap low dikes that may be flooded once in 10 years. The present main dikes, farther back, will remain as a second line of defense.

In the early stages of improving conditions for navigation, much dredging has been done at shoal places; but the writer was given to understand that, in the future, it is hoped to avoid any large amount of dredging, except at the outlet bar where fresh water and salt water meet, the current, as guided by the new training walls, being expected to shape the future channel.

Professor Giandotti, and others, stated that they were basing the designs on intensive studies of the river itself, its actual depths, and its tendencies at various localities, rather than on experimenting toward any radical changes in curvature or in design of groynes or revetment forms by means of laboratory models. This is in marked contrast to the present practice of river engineers in Germany. This attempt to confine a great meandering river to a constant width with such a minimum of shore revetment at a minimum of cost is heroic and so far as the writer knows is unprecedented.

Notwithstanding, they have been guided by centuries of observation on the behavior of this river and its neighbors, in developing their designs, the writer is confident that an adequate hydraulic laboratory could be of great aid and lead to great economics. They have yet a long way to go, and the proposed laboratory at Stra may yet be of service in their work.

In Figs. 4 and 5 are shown plans of short typical reaches, copied from maps on larger scale covering about 100 miles of the river's course that recently has been studied in much detail. The relative positions of the present main channel, the islands, sand-bars, and side channels are typical of all sediment-carrying rivers through a nearly level country.

The proposed regulated channel is indicated by dotted lines, which show a narrowing at reversals of curvature between the bends. All the water is to be concentrated in one channel, by stopping off the lesser branches of the "braided stream" at their points of divergence.

Bank revetment is indicated where the channel is close to the main dike and at all concave banks of bends. In many places the location of the main dike bears little relation to the present main channel. The width between them varies (Fig. 5) from hardly more than ¼ mile near the confluence of the Mincio to 2½ miles at a location 6 miles up stream. Since the ground for miles around is an alluvial deposit of substantially the same texture, these relative locations of channel and dike are accidental and matters of ancient history.

Similarly, Fig. 4 shows details of the position of spur-dikes for confining the water to a single channel in a typical section. The current "retards" are simple lines of piling, against which floating rubbish lodges in time of flood, thus retarding the current and inducing deposits of sediment. As stated elsewhere, it is found necessary to strengthen the up-stream retards by cross-bracing, and, in some places, by spur-dikes armored with stone rip-rap. These pile retards are in purpose very much like the permeable dikes tried unsuccessfully on the Mississippi River many years ago.

Brief Extracts from Report of March, 1924

In the report dated March 17, 1924, to the Minister of Public Works, Gabriello Carnazza, Deputato al Parlamento, by Engineer Leone Roman-

Jacur, Senatore del Regno, on the improvement of water communication from Milan to Venice, which makes, with its appendices, a volume of 236 pages, there is much of interest to engineers. Space permits only very brief extracts. On pages 45 to 51 are a series of questions and answers from which the following positive statements are condensed:

(1) The pile retards exposed to strong currents have suffered injury, and for permanence require the protection of rip-rap. Those not exposed to strong currents have endured.

(2) The effect in increasing depth of channel up to that time, when only a small part of that proposed had been installed, had not been specially noteworthy.

(3) The safety of the main dikes had not been impaired by the obstruction to river flow caused by the pile retards.

(4) From this experience it was deemed advisable to extend the works, but to limit the simpler form of pile retards to places not exposed to strong currents. Experience on the Tiber, Reno, Elba, and Vistula Rivers gave hope of success on the Po.

(5) The recent studies showed an average discharge of sediment carried by the Po, of about 10 000 000 cu. m. per year. (Equivalent to about 8 300 acre-ft., or 13 ft. in depth on 1 sq. mile.)

(6) This amount of material on its way to the sea impedes navigation by forming sand-bars where the river is broad at time of low water; and there are excellent reasons for expecting improvement to follow the proposed narrowing and the consequent increase of velocity, at low water.

(7) Until means were lacking for confining the river, dredging was necessary. This was begun in 1910.

(8) The permanence of cuts made by dredging is dependent on the material forming the river bed. There is a transition zone between Piacenze and Pavia, from gravel to fine sand, where the surface of the river bed changes.

(9) Dredging, in the course of time, may be greatly reduced, particularly after reforesting the basins of the Apennine torrents. Works for regulating these torrents and preventing erosion along their course will greatly aid the works of regulation along the River Po.

(10) From the studies of more than a century, the bed of the River Po presents no tendency to progressive rising. The sediments brought in by the torrents merely form temporary obstacles to navigation. Dredging will rapidly remove these obstacles, and, in course of time, after regulation and narrowing, the River Po may be expected to clear itself.

On pages 60 to 62, the conclusions of the Commission for various reaches of the river, are summarized as follows:

(1) From Muncio to Carnavalla-Po, a small amount of dredging suffices to maintain channel depths for 600-ton barges save at a few places presenting exceptional conditions; here regulation works are needed. For the Adda-Muncio Tract, regulation works are needed to supplement dredging.

(2) Trials on the most difficult section of the River Po, from Taro to Costello, instituted first of all from theoretical considerations, and without previous experimental data, seem to have been not altogether successful, but to have pointed the way toward improved structures promising success.

(3) The conclusion seems to have been reached by the Commission, that navigation interests could be better served by regulating the Po for barges of 600 tons capacity, than was possible by means of a parallel system of canals, which would accommodate barges of only 250 tons, and that the river can be made a fine waterway from Venice to Milan similar to the principal rivers of France and Germany.

In pages 81 to 85 there is presented a summary of the study of the regulation of the Po at low water, by the Hydrographic Office of the Po. Many of the facts therein stated have been presented in the preceding pages. The following, however, seems worthy of special attention at the risk of repetition.

The tributaries of the Po comprise three groups of very different characteristics:

(1) Torrential rivers from the Apennines, coming from the south in rapid impetuous floods, carrying much sediment.
(2) Torrential rivers from the Western Alps, similar to Group (1), but with smaller precipitation and, therefore, bringing less sediment.
(3) Those from the Alps, coming from the north, which flow through lakes and, with the exception of some small tributaries, deliver water free of sediment.

The floods in these groups come at different times, those from the Apennines first. The River Po commonly presents two low-water stages. The earlier Apennine floods leave obstructing sediments to be scoured out by the later floods of less turbid water from the north.

Cross-sections measured 10 years ago show no noteworthy change in area for low water, from those measured 50 years ago.

Measurements of average low-water elevations carried on throughout a century at Pontelagoscuro show undulations, but neither a progressive rising or lowering, thus proving no important change in elevation of river bed due to sediments unless it be that the climate and run-off have changed, which records disprove.

The turbidity measurements in the 7 years, 1914 to 1922, indicate an average annual transportation of 23 380 tons, or 17 450 cu. m. of silt, equivalent to 14 400 acre-ft. per year. (Note the estimate on page 983 of 8 300 acre-ft.)

Lombardini, more than a century ago, from the advance of the delta and the depth of deposit, estimated 27 000 000 cu. m. per year, of sedimentary material brought down by the Po, equivalent to 22 000 acre-ft. per year.

Marinelli from delta extension, from 1823 to 1893, estimated 29 000 000 cu. m., equivalent to 24 000 acre-ft. per year, as an average for the previous 70 years.

Also, some interesting details of the progress of the regulating works are given, pages 205 to 217 of the report just quoted, which are abstracted as follows:

The Minister of Public Works (Director General of Hydraulic Works) ordered the superior officers of the Civil Engineering Corps at Parma to present projects for increasing the navigation of the Po so as to permit the passage of barges of 600 tons at low water from the confluence of the River Adda to that of the Carnevalla-Po. The general plan is to unite all the low-water flow in a single channel with an area of cross-section sufficient to avoid too high a velocity of current for convenient navigation, while high enough to prevent obstruction by deposits of sediment.

The low-water discharge for planning the works was assumed to be 400 cu. m. (nearly 1 500 cu. ft.) per sec., and the slope, 0.00018 or 0.93 ft. per mile in the 26 km. (16 miles) from the confluence of the Taro to that of the Erza, and 14 km. (9 miles) farther to that of the Crostolo.

One basic criterion of design was that the equilibrium of the river must not be disturbed at either high or low water.

In order that discharge at high water may not be hindered so as to raise the flood level, the new works were all to be designed with their tops not far above the low-water elevation.

In order to avoid changing the low-water regimen of the river, the regulated channel has been designed to have an alignment near that of equilibrium, as shown by the course of the low-water channel in recent years.

In prescribing the shape of the succession of curves and the straight reaches between them, attention was given to the rule deduced by French engineers, that the greater the regularity and continuity in varying the curvature of the concave bank, the greater the regularity of variation in depth. A parabolic curve was adopted as offering at point of tangency, less discontinuity than a circular path. A surface width of 250 m. was adopted for the straight courses and increased to 300 m. at the apex of curves to provide for deposits of silt at the convex shore.

Two types of works were adopted: (1) At concave shores, protection works and revetment; and (2) for contracting and concentrating the low-water current, systems of piling retards and groynes and rip-rap are to be constructed in the convex and straight portions. These are designed to increase the deposits of sediment and ultimately to close all minor channels.

It was recognized that works directly opposed to strong currents, would have to be made of greater strength than works in more quiet waters, the piling at such places being strengthened by ties between them.

Construction was begun on the current retards during periods of low water. These consist of piles about 8 in. in diameter 18 ft. long, sunk about 12 ft., with their tops slightly above mean low water.

Lattice work was constructed between the piles at first, but its use was abandoned later, because of the belief that at time of flood, the drift of weeds, etc., would lodge on the piles and serve the same purpose. Later, however, in order to hasten the deposit of silt, the use of lattice work was resumed.

Various directions of the lines of piling in relation to direction of current were tried. Trials showed that the piling was not injured where currents were sluggish, but that strong support for it was needed against swift currents. In general, methods and materials previously used in these localities were adopted as far as practicable.

Costly excavation by mechanical means was avoided, by inducing the current to do the work, until the desired position for the concave bank was attained.

The works completed in 1922 and 1923 and up to the date of this report, comprise but a small portion of all that is contemplated. Progress has been delayed by lack of funds, but the observation of the behavior under stress of the great flood of 1926, of the works already constructed, gives valuable data for future work and so far as the writer could learn this experience is favorable to the methods originally proposed. The future progress of this work and its success are matters of great engineering interest.

Navigation Locks

A very interesting two-part navigation lock of recent construction, about 450 ft. long and 33 ft. wide, with a minimum depth of 12 ft. on the sill, was

inspected on the north shore of the River Po about 8 miles easterly from the ancient City of Adria (which gave its name to the Adriatic), and 1 mile westerly from the Etruscan coast line of about 3 000 years ago that is marked by sand dunes. This lock permits ready passage of large barges from the Po, whatever its stage of flood or drought, to the great canal system leading westerly to Adria, northward to Padua and Venice, and eastward toward the sea by the abandoned Po channel of 350 years ago.

The dikes here are 23 ft. high, and this lock provides for 20-ft. lift into the Po at times of great floods. A lift of only about 5 ft. was in evidence at the time of the writer's inspection, with the Po 2.5 ft. above its lowest stage. This lift practically measures the fall in river surface of the Po in the 23 miles from this point to the Adriatic, and indicates an extreme flood slope within the Po, from this lock to the sea, of about 1 ft. per mile, and a low-water slope of roughly 0.10 ft. per mile through the lower delta.

Another lock now being built near Ferrara on the south shore, already mentioned, will give navigable connection to the River Reno.

There is a wonderful development of irrigation canals many hundreds of years old within the northern half of the valley floor. Many of these are fed from the Adda River, and interlock with navigation canals accommodating small craft, and which, in turn, interlock with main drainage canals, all in a way which the writer made no attempt to trace. Plainly, this valley was the ancient homeland of hydraulic engineering.

River-Structure Laboratories

Inquiry and brief inspections of the technical schools at Milan and Padua indicates that the hydraulic laboratory idea has not yet taken an important part in Italy in the problems of river training. This, in the homeland of the "old masters" of hydraulic science, and coming immediately after a visit to ten of the leading hydraulic laboratories, and conferences with a dozen or more of the leading hydraulic engineers, of Germany and Switzerland, seemed unfortunate, in view of the great activity in development of water power and in the reclamation of salt marshes that is now going on in Italy, and the evident present deep appreciation of scientific education as a foundation for industrial development.

The writer inspected the magnificent new buildings constructed and equipped mainly from funds contributed by the industries and financial institutions of the Milan District for the Engineering College at Milan designed to accommodate 2 500 students. He also inspected the Engineering College at Padua which has accommodations for 1 000 students. At both institutions the hydraulic laboratories have small floor area and evidently are designed chiefly for use in undergraduate instruction. Those at Milan and Padua are excellent for that purpose, but they are not adapted for wide application of the doctrines of dimensional analysis, by means of small-scale models, to the great special problems of water power development, river training, and harbor design, like several of the hydraulic laboratories in Sweden, Germany, Austria, Switzerland, etc., where the designers of high dams, sluice-ways, and

harbor improvements are bringing their practical problems to the laboratory in increasing numbers year by year. That the hydraulic laboratory idea is now developing in Italy is proved by the elaborate report on hydraulic laboratories in other countries published about 1925 by Professor Ettore Scimini, of Padua. The writer was told that large laboratories had been delayed by lack of funds.

It is planned soon to build an excellent hydraulic laboratory at Stra, not far from Padua. Doubtless this laboratory will be at work helping to solve the problems of the Po long before the present extensive program is completed.

Improvements at the River Mouth

Some particularly interesting jetty work, recalling the Eads jetties at the mouth of the Mississippi, has just been begun for creating a navigation channel 12 ft. deep by 1 000 ft. wide, at the northern, principal entrance of the Po to the Adriatic. The depth and position of the mouths of the Po have changed greatly during the historic period (Fig. 12). At some early, perhaps prehistoric, time the main river doubtless discharged to the northeast toward Venice. About 2 000 years ago, the river easterly from Ferrara turned southward through a channel the remains of which is called the Po-Morto, or "Dead Po", and for many years was depositing its sediments in the lagoons at the northern end of the Gulf of Comacchio where maps show many remains of ancient channels diverging from the present river not far from Ferrara. Later, the river turned northward. About 300 years ago, the commercial interests of Venice feared that the mud brought down by the Po and being deposited in the Adriatic by this northern exit, would threaten the depth of navigation available to the ships of Venice. Therefore, they closed this north-flowing principal mouth. This closure turned the chief discharge of water and sediment to the east, not to the extreme south as of old, which perhaps they had expected. The channel improvement by jetty and dredging is at this present principal mouth. This is said to carry 40% of the entire discharge, the remainder being divided through several smaller mouths farther south (Fig. 12), that appear to show no tendency to enlarge.

Although the average depth of the Po for many miles up stream from the bar is said to be from 15 to 20 ft. at low water, for a considerable width of the channel, with greater depth in pools at concave bends, the channels at the junction of turbid fresh water with salt water suddenly shoal to a depth of only 2 to 4 ft. They also spread out over the bar to a much greater total width than up stream. This shoaling extends perhaps half a mile or a mile along the river's course, with very gentle slopes in both directions to and from the crest of the bar.

For defining the new channel across the bar, one long, straight training wall somewhat similar to that shown at (*E*), Fig. 9, but with a larger proportion of rubble stone, is now being built at a slight angle to the approaching current, the impact from which should induce a deep channel near the wall. Several long, narrow cuts have been dredged across the bar within the future channel in order to determine its composition and hardness, and perhaps to invite erosion.

There is a strong hope that the construction of a second parallel training wall can be deferred, although the plans provide for it. Later, it may be found expedient to build obstructions across some of the other mouths of the Po so as to force a greater current through this principal mouth, but this is now believed improbable, because for some years past the present chief outlet has shown a tendency to increase in size.

Advance of the Po Delta

The steady advance seaward of the shore line of the Po Delta into the Adriatic during the past 2 000 years to the extent of more than 15 miles at the outward corner of its fan-shaped delta, which is about 50 miles broad from north to south, is both of historic and practical interest (see Fig. 12). To the writer this increased length of river appears an important factor in raising the bed and the low-water surface of the Po in its winding pathway through the lower valley, probably more important than any deposit of sediment caused by confining the floods between dikes. At a point which 1 000 years ago marked the river mouth at sea level, the river surface at times of high flood now stands 10 to 15 ft. higher than before and this must affect the height of the river bed and the height of the water surface far up stream.

A rough measurement of the effect of this pushing out of the delta in raising the elevation of the water surface, and perhaps, also, that of the river bed, was found in the reading of the Po River gauge at the lock near Donada, described previously. This stood at the equivalent of 4.3 ft. on a gauge said to have its zero at sea level which, neglecting tides, indicated a fall or slope of 4.3 ft. in about 20 miles of river seaward, while discharge was such as to raise the river here about 1.3 ft. This indicates a low-water slope of about 3.0 ft. in 20 miles, a large part of which may have been over the bar at the river's mouth.

The newly made delta surface between the mouths appeared in general to be 1 to 1.5 ft. above sea level, explainable by the deposit being largely made from water spreading out in time of high flood and being caught in the grass and bushes of the marsh. A low reclamation dike is being thrown around the newly formed land. For the past two or three centuries the accretion has averaged about 300 acres per year.

Successive advances of the shore line are shown on the accompanying map, Fig. 12, traced from a recent study given the writer at Venice by Luigi Miliani, Director of the Works for the Lower Po. The shore line of the Adriatic in the days of the Etruscans, about 3 000 years ago, is marked by a long north-south line of low sand dunes rising 10 to 20 ft. above the level of the plain. The writer visited these dunes but found no reason that explained why they marked the shore line of that particular stage of development, and are not found along the later stages of its advance, other than that these sand dunes may have been the product of winds blowing landward across a sea beach which maintained a nearly constant position during many centuries, while the river was discharging southward through the Po di Primaro, filling with sediment an ancient indentation of the coast at the peak of the Gulf of Comacchio.

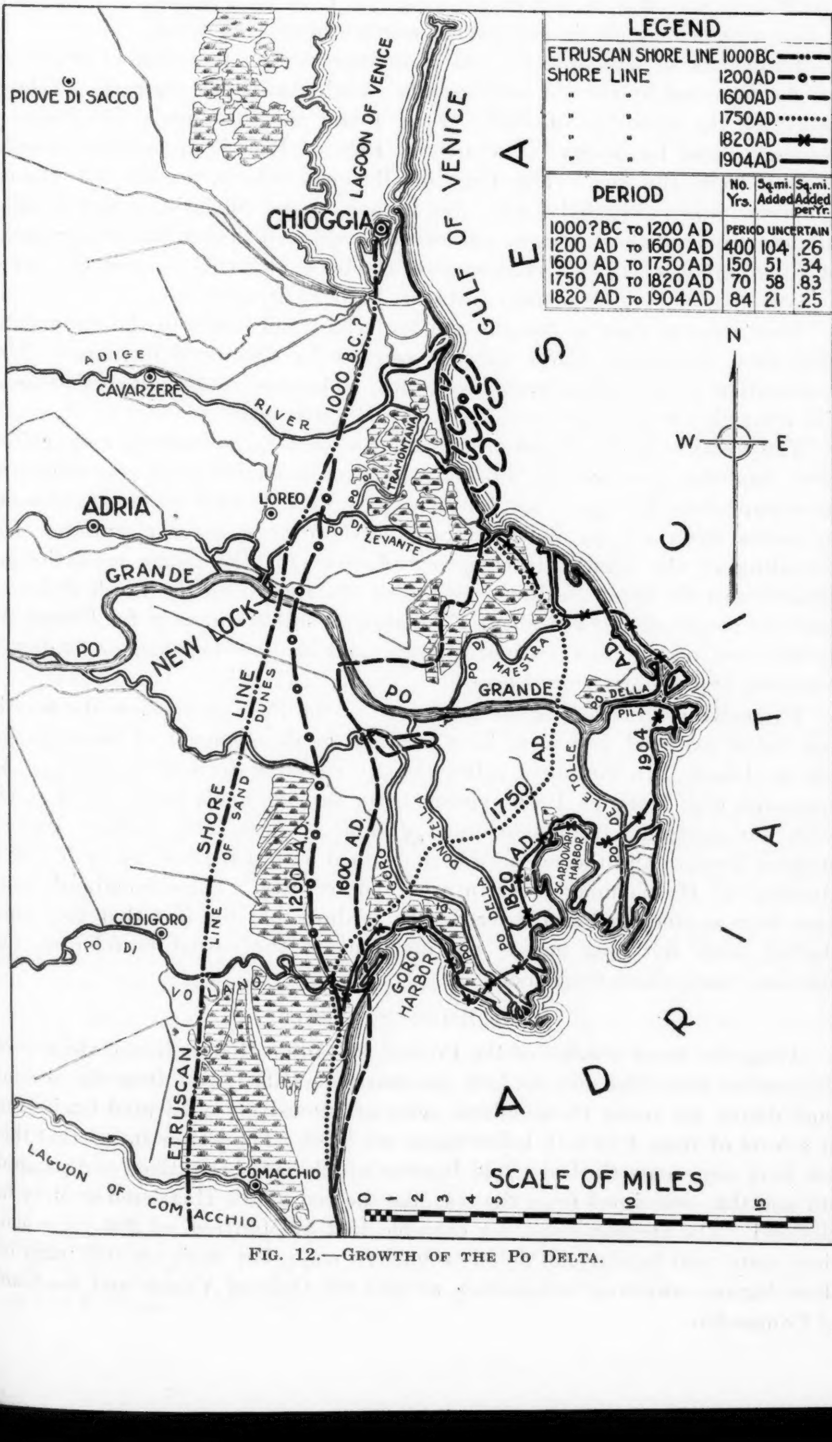

FIG. 12.—GROWTH OF THE PO DELTA.

The tides in the Adriatic, other than wind tides, are only about ½ ft. in range. A wind tide may lift the water as much as 5 ft. The pavement in the Piazza San Marco, in Venice, is said to have been flooded to a depth of a foot after a long-continued, strong, northward-blowing wind.

Extensions of land into the sea at the mouths of other Italian rivers are briefly discussed by the old authors, who sought in littoral currents, explanations for the direction of drift. Frisi found this principally in Tuscany, Romagna, and La Marca. The ancient Port of Pisa is at present 10 miles distant from the sea. The City of Ravenna which, under the Roman Emperors, was celebrated for its fine harbor on the Adriatic, is now 5 miles inland. Some of these ancient authorities concluded that to insure the safety of a sea harbor a sediment-bearing river should be diverted to enter the sea at a point at least 8 miles distant from the harbor entrance.

Vast areas of shallow lagoons and low-lying mud banks in the outer delta have been reclaimed within the past century by dikes and drainage. This reclamation is still being pushed outward. There is nothing now that looks like a marsh, except a narrow fringe near the outer edge.

The treatises of Frisi and others indicate that many swamps and shallow lakes like that now around Mantua, which existed midway of the valley far up stream from Ferrara a few centuries ago, were drained by canals running to points down stream along river or seashore at a lower elevation. The providing of the present intercepting channel for the many north-flowing streams easterly from the Reno, by taking the Reno into the Po di Primaro, much as proposed by Frisi about 1760, appears to have greatly facilitated the reclamation of the vast area of swamps and shallow lagoons, or "valleys", southerly and easterly from Ferrara.

Extensive reclamation in the lower part of the Po Valley where the land is now below sea level, is said to have awaited the development of steam pumping machinery. In the upper valley swamps could be drained by gravity. As compared with Holland, Italy appears never to have made large use of windmills for pumping from submerged ground. Large, old, shallow lagoons between Ferrara and the sea have been drained within the past 50 or 75 years, rivaling the Haarlemmer Meer in area and fertility. These reclaimed lands have been so smoothed and covered with verdure that the fact that they were shallow lakes not long ago is not apparent to the casual traveler on the excellent roads which cross them.

Land Below Sea Level

Along the lower reaches of the Po and the Adige Rivers, in the vicinity of the ancient shore line and the lock previously described, west from the ancient sand dunes, are many thousands of acres of intensively cultivated land lying at a level of from 1 to 4 ft. below mean sea level. The writer infers that this low level represents the bed of old lagoons which have been diked and pumped out and thus reclaimed from the sea, like the bed of the Haarlemmer Meer in Holland. The ancient maps, for example that of Manfredi of 200 years ago, show many vast lagoons not noted on modern maps, but there are still many of these lagoons awaiting reclamation around the Gulf of Venice and the Gulf of Comacchio.

Local subsidence of the earth's surface as a cause for depression of these lagoons was suggested by authors of one or two centuries ago. Frisi, in his fifth chapter, discusses this in terms of the rising of the waters rather than of subsidence of the land. He says the continual rising of the waters was not unknown to the learned of the Sixteenth Century. Eustace Manfredi established the fact at Ravenna, a few miles south of the mouth of the Po, by levels on floors of several ancient edifices which he found below the level of the sea. Bernadin Zendrini confirmed this at Venice, where rings formerly used to fasten boats are now below the level of the sea, and the subterranean church of St. Mark is no longer serviceable. Lest this be thought to be due to settlement of the foundations, he cites observations on the opposite side of the Adriatic, where the sea is now higher than the floors of ancient buildings built on solid rock.

Naturally all this was of interest to the writer because of his recent investigation of earth tilt in the Great Lakes region in the United States and his discovery of 25 years ago, from old bench marks and tide records, that subsidence at Boston, Mass., had been going on at the rate of about 1 ft. per century;* also, because of finding in the course of studies for the New York water supply, about 23 years ago, that the tide-gauge records near New York, N. Y., indicate subsidence at the rate of about 0.6 ft. per century.

River Bed Higher than Adjacent Land

At the beginning of this paper the oft-repeated statement that the elevation of the bed of the River Po is higher than the land on either side, was mentioned. It is stated in the Encyclopædia Britannica that:

"Owing to its confinement between these high banks and to the great amount of sedimentary material which the river brings down with it, its bed has been gradually raised so that in its lower course it is in many places above the level of the surrounding country. A result of confining the stream between its containing banks is the rapid growth of the delta."

The writer was unable to learn of any valid complaint against the dike system of the River Po on this score. The statement appears to be untrue.

The report to the Ministry of Public Works, previously quoted, states most distinctly, on page 50:

"As seen from the said studies of the Hydrographic Office of the Po and from the hydrometric observations of the Po for a century, it is evident that the bed of the river presents no tendency to a progressive rising."

It is probable that on the tributaries of the Po, at the outer edge of the nearly level valley floor, immediately down stream from the steeper gradient of the river within the hills, there may be important deposits within the dikes which have raised the beds of these tributaries above the level of the land outside the dikes. The writer has seen examples of this on streams in Korea, but in this brief inspection no place could be found where the bed of the Po was at a higher level than the banks outside, nor could it be learned from the

* See Appendix No. 20, Report by John R. Freeman, Past-President, Am. Soc. C. E., to the Committee on the Charles River Dam, Commonwealth of Massachusetts, 1903.

† See p. 653, Report of the Commission on Additional Water Supply for the City of New York, 1904, by William H. Burr, and the late Rudolph Hering, Members, Am. Soc. C. E., and John R. Freeman, Past-President, Am. Soc. C. E.

engineers of any such place along the main channel of the Po down stream from Cremona. Professor Scimemi, Chief of the Department of Hydraulic Engineering, in the Technical University at Padua, has written that he knows of no such instance from Mincio to the sea..

On the Adige River and other streams immediately north from the Lower Po, there are places where the bed is higher than the adjacent land. The Adige, unlike the Adda, Ticino, and Mincio, has no large lakes in its course. A cross-section illustrating this was traced for the writer in the engineering office at Rovigo, which is presented in Fig. 13. This is located about 22 miles up stream from Rovigo, or about 55 miles up stream from the mouth of the river. It shows the river bed 2.3 ft. higher than the land outside, and the water in the river 22.3 ft. above the land. Presumably, this was at ordinary stage and flood stage only 2 or 3 ft. below the top of the dike. In some other rivers, the bed is several meters higher than the land outside.

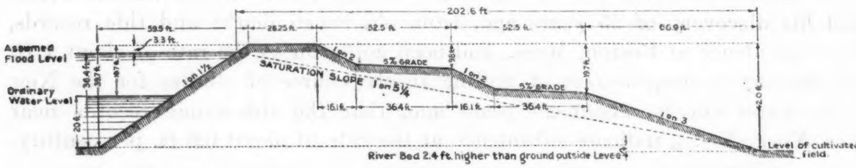

FIG. 13.—TYPICAL SECTION OF LEVEE ON RIGHT SIDE OF THE ADIGE RIVER.

On the Reno River where it leaves the steep slope within the hills southwesterly from Bologna, one would probably find similar examples. Time did not permit a tour along both the Reno and the Adige. The writer is confident that tracing these rivers from the sea, back into the slopes within the foothills, would disclose much of great interest to an engineer.

In closing this narrative it is suggested to the engineer tourist that in addition to the art treasures of Florence, the marvels of cathedral architecture, or the mysteries of the Etruscan hill-cities, he can find in Italy plenty, both ancient and modern, that is of surpassing interest in the line of his own profession. The books of the "Old Masters of Hydraulics" also are fascinating. One who has scant knowledge of the language can translate much from analogy to the French and Latin of schoolboy days. The writer is trying to enlist the aid of some Italian engineers, including Lorenzo Allievi and Gaudenzio Fantoli, in preparing abstracts of some of these old treatises, which will trace the development of this branch of science and art in its early home for the benefit of English-speaking engineers. The Italian backgrounds of hydraulic engineering present some very interesting scenery.

All that was seen and heard, of these works of river improvement, reclamation of marsh lands along the Po and farther south, and of the great activity in hydro-electric development of the highest order of excellence on Alpine streams, as well as the enterprise in technical education, the encouragement of general international scientific societies to come to Italy for their conventions in the year just past, indicate that Italy has taken a vigorous new departure toward prosperity, largely based upon modern science.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

ONE HUNDRED FIFTY YEARS ADVANCE IN STRUCTURAL ANALYSIS*

By H. M. Westergaard,† M. Am. Soc. C. E.

Synopsis

The story of the advance of structural analysis is, in the main, an account of the work done by individual men. While the science of structural analysis is international, at the same time its national traditions are significant. In the French tradition the names of Coulomb, Navier, and Saint-Venant stand out; in the English tradition the name of Rankine; in the German the names of Mohr, Müller-Breslau, and Föppl; and there is a Russian tradition which may be traced back to the influence of Euler.

The account which follows begins, somewhat arbitrarily, but (as will be seen) not entirely without justification, with the work of Coulomb. Consideration is given, however, to his predecessors. An attempt is made to sketch the development from this early period to the present.

The ancient bridges, the vaults, arches, and flying buttresses of the great medieval cathedrals, and the riggings of sailing vessels at the height of their development, bear witness of structural insight on the part of the builders; but the art of structural analysis, except for small beginnings, is only about 150 years old.

Structural analysis is based on the physical properties of materials; its method is mathematical; its purpose is design. It came into existence by a meeting of physics, mathematics, and engineering.

Coulomb, 1776

In 1776, the Paris Academy of Sciences published a paper by Charles Augustin Coulomb, entitled, "Essai sur une application des règles de Maximis

Note.—Discussion on this paper will be closed in August, 1928.

* Presented at the meeting of the Structural Division, Philadelphia, Pa., October 8, 1926.

† Prof. of Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.

et Minimis à quelques Problèmes de Statique, relatifs à l'Architecture".* Coulomb, who lived from 1736 to 1806, was a military engineer. He possessed mathematical insight and attained distinction as a physicist in the fields of mechanics, electricity, and magnetism. Discussing beams with rectangular cross-section, in the paper of 1776, he assumed the fibers to offer resistance proportional to their extension or shortening; he considered the balancing of the internal forces acting upon a cross-section; and he determined, by accurate reasoning, the position of the neutral axis and the moment of the internal forces. He remarked that at rupture, in some cases, the neutral axis may be at a different position. In the same paper, he considered the deformation by shear in connection with the failure of a solid; and he presented his theory of earth pressure on a retaining wall, according to which a wedge of earth slides when the friction and cohesion in a plane section become insufficient.

Saint-Venant remarked eighty years later† that in this paper "one finds presented almost all the bases of the theory of stability of structures." If one considers the combined circumstances of Coulomb's capacity for exact science and his early relation to practical affairs of engineering, it is not strange that he should be the author of so significant a document.

It is perhaps stretching a point to state that, because of this paper, Coulomb is the father of structural analysis. He certainly is one of the fathers.

Coulomb's Predecessors

It would be unreasonable, however, not to consider Coulomb's predecessors.‡ Galileo, in a dialogue published in 1638, inquired into the problem of the strength of a cantilever. He considered the beam to be perfectly rigid except for the turning about a horizontal axis in the section of rupture. In spite of certain incorrect assumptions he obtained a correct design of a cantilever of uniform strength for the case of a rectangular cross-section. Hooke, in 1678, announced his famous law that deformations are proportional to the loads. Mariotte at about the same time arrived at the same law, and applied it to the fibers of a beam. Tests (made by him in 1680) led him to the observation that some of the fibers of a beam are stretched and others shortened. He placed arbitrarily the boundary between extension and shortening at the middle of the beam. Varignon, to whom credit belongs for early developments in statics§ (for example, the polygon of equilibrium of a string), discussed the investigations of Galileo and Mariotte.‖ Varignon computed a moment of resistance of a cantilever by assuming the tensile force of the fibers to be proportional to the distance from an axis, but, unfortunately, in spite of

* In the volume for 1773 of "Mémoires de Mathématique et de Physique, Présentés à l'Académie Royale des Sciences, par divers Savans, et lus dans ses Assemblées, pp. 343-382.

† *Journal de Mathématiques Pures et Appliquées,* 2.Série, v. 1, 1856, p. 90.

‡ A very complete account of the development of ideas and methods relating to the theory of elasticity and strength of materials may be found in the great work of Todhunter and Pearson, "A History of the Theory of Elasticity and of the Strength of Materials," Cambridge Univ. Press, 1886–93. A shorter account is given by A. E. H. Love in the historical introduction to his "Mathematical Theory of Elasticity," Third Edition, 1920. See, also, H. Lorenz, "Technische Elastizitätslehre," 1913, pp. 644-683.

§ P. Varignon, "Nouvelle mécanique ou statique," 2 vol., Paris, 1725.

‖ In a paper published in the *Mémoires* of the Paris Academy, in 1702.

Mariotte's observation of shortened fibers, he placed this axis at the bottom of the cross-section.

James Bernouilli, the earliest member of the distinguished Bernouilli family to attain fame as a mathematician, at the end of the Seventeenth and the beginning of the Eighteenth Century introduced the problem of determining the shape of the elastic curve. After him is named the assumption that a plane cross-section of a beam remains plane after bending. He arrived at the peculiar, inadmissible conclusion that the position of the neutral axis is indifferent. His nephew, Daniel Bernouilli, about 1735, obtained a differential equation for transverse vibrations of a bar.

Leonhard Euler was born in 1707 in Basel, Switzerland, the city of the Bernouilli's. In 1727 he was called to the St. Petersburg Academy by Catherine I, and in 1741 to the Berlin Academy by Frederick the Great. He returned to St. Petersburg in 1766 by invitation of Catherine II, and died in 1783. He was endowed with a magnificent intuition, and was one of the great mathematicians of all times. The Bernouilli's influenced him to attack the question of the elastic curve. In order to understand his method, it should be noted that the science of physics in the Eighteenth Century was still struggling to emancipate itself from metaphysics. In a manner characteristic of this state of development, he argued* from the premise of the perfection of the universe to a principle of least action. In this way he established a principle of minimum for the flexure of beams, and he gave solutions for a number of cases. In a later paper, of 1757, entitled "Sur la force de colonnes",† he derived the celebrated formula which is named after him, and which expresses the critical load at which a slender column buckles.

In this early work on the elastic curve of beams and columns the quantity which is now represented by the product of the modulus of elasticity and the moment of inertia was given as a single "moment of stiffness", characteristic of the beam or column. Euler suggested determining experimentally this moment of stiffness ("moment du ressort" or "moment de roideur") by supporting the beam or column as a cantilever and measuring the deflection at the end, due to a transverse force at the end, a case in which the deflection could be obtained by a simple formula.

In addition to his work on the static elastic curves, Euler, during his later years, investigated successfully the transverse vibrations of beams, and, by an attempt to analyze the vibrations of bells, he began the work on the two-dimensional problems of plates and shells.

Progress and Relapses, 1776-1820

Consider again the paper by Coulomb, published in 1776. In spite of the unquestioned importance of the work of his predecessors, especially that of the great mathematician, Euler, Coulomb's paper stands out as a most important original document produced by an engineer, that deals with structural analysis for the sake of the structure. This paper does contain, besides

* Euler, "Methodus inveniendi lineas curvas maximi minimive proprietate gaudentes," Lausanne, 1744, Additamentum I: "De curvis elasticis."

† Published in the *Mémoires* de l'Académie de Berlin, v. 13, 1759.

other important material, the first adequate analysis of the fiber stresses in a beam.

It must not be thought, however, that Coulomb's analysis of the beam at once settled this issue in the minds of all others who were concerned with it. Girard published, in 1798, a treatise on the resistance of solids.* A German translation was published in 1803. This book is the first treatise on mechanics of materials. It has a historical introduction, and contains extensive accounts of the earlier works from Galileo to Euler and Lagrange. It reports tests performed on wooden beams by the author, and it is significant on account of the importance attached to experiments. Tests are in, and metaphysics is out. Girard, however, accepted James Bernouilli's incorrect conclusion that the position of the neutral axis is indifferent.

The learned Dr. Thomas Young, famous as a physicist and a philologist, made two lasting contributions to the subject of elasticity.† One is the discovery of the detrusion, or shearing deformation, as an elastic deformation (Coulomb had considered it as a permanent set in connection with failure); the other is the introduction of the modulus of elasticity, which is often named after him. Young, still, was uncertain and obscure as to the position of the neutral axis.

The textbooks from the beginning of the Nineteenth Century, in dealing with the subject of the strength of beams, are not very satisfactory. Olinthus Gregory, in his "Treatise of Mechanics, Theoretical, Practical, and Descriptive", London, 1806 (and in the later edition, 1815), accepted Galileo's results, because they are simple, and ignored the others. Todhunter and Pearson‡ refer to this book as evidence of "the depth to which English mechanical knowledge had sunk at the commencement of the nineteenth century." The book by John Banks, "On the Power of Machines" (1803), contains practical rules and erratic theory for the design of beams. Eytelwein,§ who possessed better mathematical understanding than his two contemporaries just mentioned, introduced the subject of beams well, but fell into the old error of placing the neutral axis in the concave surface of the beam.

In France, Duleau in his "Essai théorique et experimental sur la résistance du fer forgé" (Paris, 1820), quoted Coulomb on the subject of the neutral axis, but misunderstood him. In England, Tredgold in his two standard textbooks, "The Elementary Principles of Carpentry" (London, 1820), and "A Practical Essay on the Strength of Cast Iron" (London, 1822), displayed confusion in matters of theory. Hodgkinson, an able experimenter, contributed toward correcting the current errors about the neutral axis, especially through two papers published in 1824 and 1831 by the Manchester Literary and Philosophical Society. Barlow was erratic on the subject. In the edition of 1837 of his "Treatise on the Strength of Timber and Cast Iron,

* P. S. Girard, "Traité analytique de la résistance des solides, et des solides d'égale résistance, auquel on a joint une suite de nouvelles expériences sur la force, et l'élasticité spécifique des bois de chêne et de sapin," Paris, 1798.

† "Course of Lectures on Natural Philosophy and the Mechanical Arts," Lond., 1807.

‡ "A History of the Theory of Elasticity and of the Strength of Materials," 1886–93, v. 1, p. 88.

§ "Handbuch der Statik fester Körper," Berlin, 1808.

Malleable Iron and Other Materials" (page 63), he finally adopted Coulomb's principle for determining the neutral axis. He credited this principle to Hodgkinson.

The Founding of the Theory of Elasticity, 1820-1830

The picture of advance in structural analysis again becomes cheerful when one turns the attention toward France during the decade from 1820 to 1830. Three names stand out: Navier (1785-1836), Cauchy (1789-1857), and Poisson (1781-1840). These three men are considered to be the founders of the theory of elasticity. Navier, who became Professor of Mathematical Analysis and Mechanics at the École Polytechnique in Paris, combined ability in mathematics with competence in practical affairs of engineering. Cauchy entered the École Polytechnique in 1805 and the École des Ponts et Chaussées in 1807, and began a career as an engineer. In 1813, however, the two great mathematicians, Lagrange and Laplace, influenced him to renounce engineering for mathematics. Poisson, like Navier, was a Professor at the École Polytechnique. He was an ingenious and successful mathematician. His interests were directed toward pure science rather than toward its applications.

It was in 1821 that Navier submitted to the Paris Academy the investigation by which he originated the theory of elasticity of three-dimensional solids. By considering forces acting between the molecules according to a definite law of attraction and repulsion he established, for the first time, a set of equations for the equilibrium and the vibrations of the interior parts of a solid. At about the same time interest in the questions of waves of light aroused Cauchy to a study of the mechanics of an elastic medium. He succeeded, before the end of 1822, in setting up the fundamental relations of elasticity (in the form now current), in terms of the principal directions of stresses and strains, the six components of stresses and strains, respectively, and in terms of differential equations for the three components of displacement. Cauchy, later, added studies of the elasticity of crystals, which, on account of different properties in different directions, may have as many as twenty-one independent elastic constants (that is, constants of the nature of Young's modulus or Poisson's ratio). Poisson's most important contribution was presented to the Paris Academy in 1828*. This paper, besides fundamental theory, contains numerous specific applications.

It should be remarked that the theory of elasticity is primarily physics, aimed at the understanding of matter. The development of the fundamental processes of theory, through the past one hundred years, has been the joint work of physicists, mathematicians, and engineers. Applications have presented themselves to molecular theory and theory of sound. At the same time, applications to structural analysis have been a cause of continual contact with engineering. These practical applications to engineering have come into the foreground during more recent years.

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* Published in the *Mémoires*, v. 8, 1829, pp. 357-570.

Mechanics of Engineering According to Navier

Navier, scientist, engineer, and professor, became the author of the first great textbook on mechanics of engineering. The first edition of his "Résumé des leçons données a l'École des Ponts et Chaussées sur l'application de la Mécanique a l'établissement des constructions et des machines", was published in 1826, and the second, revised edition in 1833. The book appeared also in an Italian translation. It is evidence of the great influence of this book, and of the high esteem in which it was held, that Barré de Saint-Venant, one of the great classics in the theory of elasticity, undertook, a number of years after the death of Navier, the preparation of a third, annotated edition, which appeared in final form in 1864.*

Navier, in the preface to the first edition, expressed well the purpose of structural analysis by comparing the stagnancy resulting from design by imitation with the progress which scientific methods make possible. The book, eminently practical, draws extensively on the experimental material available at the time. Of importance is the treatment, new because of its completeness, of the subject of beams. Stresses are determined for any cross-section, and the deflections are found by the method of double integration. This treatment is based on Bernouilli's assumption, often named "Navier's hypothesis," that the plane cross-section remains plane after bending. It was an accomplishment of investigators in the theory of elasticity of a later day, especially Saint-Venant and Pearson, to show that the errors introduced by this approximate assumption are only small for beams of ordinary dimensions. The result is that Navier's treatment of beams has become justly traditional. Although some additions have been made to it, this treatment holds its place in structural analysis in engineering of the present day.

Navier dealt competently in his "Mécanique" with subjects such as earth pressures, stability of masonry arches, and timber structures.

He treated particular cases of continuous beams, and he gave an analysis of the deflections of curved beams with a not too sharp curvature. He applied the latter analysis to some cases of two-hinged elastic arches. In 1825, Navier had published a note on the statics of bodies supported at more than three points.† He made therein the observation that a statically indeterminate problem becomes statically determinate if one takes into account conditions of elasticity. Except for some cases of statically indeterminate beams, which had been analyzed before, this note, in connection with the several cases treated in the book, represents, as far as is known, the beginning of the analysis of elastic, statically indeterminate structures. The same problems can be solved more easily by modern methods, but Navier's solutions are legitimate.

Navier applied the assumption of the plane cross-section remaining plane to torsion. This is proper in the case of a circular section, but improper in the case of non-circular sections. It was Saint-Venant who accomplished the

* This third edition, besides the additions to the text, contains an account of the life of Navier and a general historical introduction.

† Navier, "Sur des questions de statique dans lesquelles on considère un corps supporté par un nombre de points d'appui surpassant trois," *Bulletin des Sciences* à la Société Philomatique, 1825, p. 35.

difficult task of producing an exact theory of torsion for non-circular sections, and he inserted this theory in the third edition of the "Mécanique".

Navier's book is written with admirable clarity. It is no wonder that it exerted a great influence.

Two more of the works of Navier should be mentioned. In 1820, he submitted to the Paris Academy a paper on the flexure of slabs. Lagrange had obtained in 1811 a differential equation for this type of flexure. Navier gave an accurate solution for the case of a rectangular slab simply supported on four sides. The other work to be mentioned is a report of 1823 on his examination of English suspension bridges. It is significant that he tried to solve problems of the vibrations of the bridge.

The French Tradition

Lamé (1795-1870) and Clapeyron (1799-1864) were French engineers, who were for a while in Russian service. They contributed some papers jointly, especially one on the theory of internal equilibrium of solids, which was published by the Paris Academy.* This paper contains a number of definite applications of the theory of elasticity. Lamé published, in 1852, a volume of lectures on the theory of elasticity.† His formulas for the stresses in thick cylinders with internal pressure are well known. Clapeyron established an important formula, named after him, for the internal energy of an elastic body, due to the stresses.

In a short paper published in 1857‡ Clapeyron presented the famous "theorem of three moments" by which continuous beams may be analyzed conveniently. The paper has the appearance of a preliminary announcement of a longer paper, which, however, was never published. The form is rather interesting. The author begins by referring to the immense capital invested in railroads and the resulting growth of the problems of the engineers. He mentions a particular bridge with continuous girders, and says that practice here, as so often, has preceded theory. He states the formula, and shows by an example how to apply it.§

The great classic in the theory of elasticity, Barré de Saint-Venant (1797-1886), has been mentioned already in connection with the third edition of Navier's "Mécanique." Saint-Venant was an engineer and a mathematician. Most famous is his work on the subject of torsion.‖ Saint-Venant gave exact solutions for a number of definite cross-sections. Knowing the needs of engineers, he did not satisfy himself with establishing procedures of computation, or solutions in terms of infinite series, but he computed numerical coefficients, and represented many of the results graphically in a manner readily intelligible to any trained engineer.

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* *Mémoires* présentés par divers Savans, 1833.

† Lamé, "Leçons sur la théorie mathématique de l'élasticité des corps solides," Paris, 1852, Second Edition, 1866.

‡ *Comptes Rendus,* Paris Academy, v. 45, p. 1076.

§ The theorem of three moments is frequently named after Clapeyron, through whom it became generally known. It was given earlier, however, by Bertot, in *Comptes Rendus* de la Société des Ingénieurs Civils, 1855, p. 278. A more general form, applicable not only to a uniform load on each span, but to any type of load, was given later by Bresse.

‖ "Mémoire sur la torsion des prismes, etc," pub. by the Paris Academy in "Mémoires des Savants Etrangers," 1855, pp. 233–560.

One may explain the existence of a French tradition in structural analysis by referring to Navier and Saint-Venant. This tradition was upheld by disciples of Saint-Venant, among whom should be mentioned Boussinesq, H. Resal, Lévy, and Flamant. The researches of Boussinesq in a variety of fields of the theory of elasticity, theoretical and applied, as well as in the theory of earth pressures, are of considerable depth. H. Resal originated in 1862 an acceptable approximate theory of sharply curved beams (or hooks)*. Lévy wrote an extensive and valuable treatise, "Statique graphique" (Paris, 1874, Second Edition, 4 vol., 1886-88). And Flamant wrote treatises on structural statics and strength of materials.†

It was in France in 1894 that E. Coignet and N. de Tédesco, in a report to the Société des Ingénieurs Civils, originated the method which is now commonly used for computing nominal stresses in reinforced concrete beams for the purpose of design. This computation is based on neglect of horizontal tensile stresses in the concrete, on a linear relation otherwise of stresses and strains, and on the assumption that the plane cross-section remains plane.

A. Mesnager is a prominent contemporary representative of the French tradition. He introduced and made practicable the analysis of a complex structure by applying polarized light to a transparent model.‡ He has made important contributions to the theory of flexure of slabs.§

England

France had Navier; Great Britain produced W. J. M. Rankine (1820-72). As Professor of Civil Engineering and Mechanics in the University of Glasgow, Rankine wrote remarkable textbooks in various fields of engineering. The first edition of his "Manual of Applied Mechanics" was published in London in 1858, the nineteenth edition in 1914. This book stands side by side with Navier's "Mécanique" as a great textbook on mechanics of engineering. New in this book is the determination of the distribution of shearing stresses in a beam by the analysis now current.|| Another mark of progress is the introduction of the formula, which has been named after Rankine, for the strength of columns. Although not perfect, this formula has been of great service. Rankine credits the formula to Gordon. It had been proposed previously by F. Schwarz in 1854.

Rankine treated the problem of earth pressures from the point of view of a plane state of stress. His solution compares in importance with that of Coulomb. The two theories agree in important cases.

Rankine wrote numerous scientific papers, some of which deal with the theory of elasticity.

To understand the English tradition one must refer to Rankine; to his predecessors, Hodgkinson on the practical, experimental side, and the great mathematicians, Green and Stokes, on the side of the mathematical theory

* *Annales des Mines*, 1862, p. 617.

† A. Flamant, "Stabilité des constructions; Résistance des matériaux," Paris, 1886; Second Edition, 1897.

‡ *Annales des Ponts et Chaussées*, 1913, p. 133.

§ A series of papers in *Comptes Rendus*, Paris Academy, 1916–19.

|| First Edition, 1858, p. 338.

of elasticity; also to two great physicists of the Nineteenth Century, Maxwell and Lord Kelvin; and to Todhunter and Pearson, authors of the monumental "History of the Theory of Elasticity" (Cambridge, 1886-93). The practical problems of structural mechanics have been represented during recent years, for example, by A. Morley, author of noteworthy treatises on "Strength of Materials" and "Theory of Structures"; and the mathematical theory of elasticity has an eminent contemporary representative in A. E. H. Love, author of a great treatise on this subject (first edition, 1892-93; third edition, 1920).

Early Works on Trusses

The prominent American engineer, the late Squire Whipple, Hon. M. Am. Soc. C. E., inventor of the Whipple truss, published a work on "Bridge Building"* in 1847. This work contains an analysis, which, so far as is known, is the first legitimate analysis of the jointed truss. His style reveals a clear mind. He accomplished his task by means of the simplest possible principle of statics, that of the force resolved into a vertical and horizontal component. The American engineer, Herman Haupt, evidently independent of Whipple, followed in 1851 with an attempt to analyze trusses.† One finds in his work rather a mixture of sound and unsound thinking.

In 1863, August Ritter, in Germany, published his famous method of sections by which each stress is found by expressing the moments of all forces on one side of the section with respect to a suitable center.‡ This method holds its place at the present day as a most effective way of determining stresses in trusses.

Graphical Methods

Graphical methods have played an important part in structural analysis. A "graphical statics" exists.

Particular graphical methods were used fairly early. Poncelet, for example, proposed, in 1840, his elegant method (frequently credited to G. Rebhann in 1871) for determining the earth pressures which exist according to Coulomb's theory. The English clergyman and Professor of Natural Philosophy, H. Moseley, a disciple of Poncelet, introduced in 1833 the line of pressure of the masonry arch.§

C. Culmann, born in 1821, was a Professor at the Polytechnikum in Zürich, Switzerland, from 1854 until his death in 1881. While Varignon in his "Nouvelle mécanique" of 1725 had studied the polygon of equilibrium of a string, it is reasonable to state that graphical statics as an independent

* See S. Whipple, "Elementary and Practical Treatise on Bridge Building," an enlarged and improved edition of the author's original work; Second Edition, N. Y., 1873. Reference is made in the preface to the "Original Essays" of 1847. See, also, H. G. Tyrrell, "History of Bridge Engineering" (published by the author, Chic., 1911), p. 166; or the historical section in the recent, very excellent book by J. I. Parcel and G. A. Maney, "An Elementary Treatise on Statically Indeterminate Stresses," N. Y., John Wiley & Sons, 1926, p. 352.

† See Herman Haupt, "General Theory of Bridge Construction," N. Y., 1869. Pt. I of this book represents Haupt's early work.

‡ A. Ritter, "Elementare Theorie und Berechnung eisener Dach- und Brückenkonstruktionen," Hannover, 1863.

§ H. Moseley, "On the Equilibrium of the Arch" (paper read 1833), *Transactions*, Cambridge Philosophical Soc., v. 5, 1835, p. 293; or "The Mechanical Principles of Engineering and Architecture," Lond., 1843, p. 403.

subject came into existence when Culmann, by emancipating himself from the idea of a material string, made the string polygon a tool of analysis. Culmann found many uses for the string polygon; for example, in the analysis of masonry arches; and in determining the moment of inertia of an area. In the analysis of trusses he introduced a method of sections similar to that of Ritter, except that the three stresses in the section are found graphically as components of the resultant of the external forces on one side of the section. Culmann's remarkable book on structural mechanics, published in 1864-66, has the title, "Graphische Statik".

J. Clerk Maxwell (1831-79), the great English physicist, invented, in 1864,* the stress diagram for trusses which unites into a single figure all the individual force polygons. Cremona developed the method further. The diagram is often named after Maxwell and Cremona. It found its place quickly in graphical statics of engineering.

Graphical statics owes a great deal to Otto Mohr (1835-1918), for a number of years Professor at the Technische Hochschule in Dresden, Germany. In 1868 he published his method of finding deflections of a beam by use of string polygons. He established on the same occasion the closely allied method of computing deflections as if they were bending moments.† The late C. E. Greene, M. Am. Soc. C. E., at the University of Michigan, about 1873, discovered a related principle of moments of areas by which one may find the deflection of any point of a beam, measured from a tangent to the elastic curve. H. Müller-Breslau, in 1885, gave Mohr's principle a general formulation,‡ with the application extended to trusses; the general principle includes Greene's principle as a special case.

Mohr invented a use of string polygons in determining stresses due to bending combined with compression when the material has no tensile resistance. C. Guidi, in Italy, showed in 1906 that this method, with a small modification, can be applied to a reinforced concrete member with any symmetrical cross-section.

Mohr devised an elegant graphical representation of the three-dimensional state of stress at a point, and he invented the circle of inertia for determining moments of inertia of an area with respect to different axes through the same point. Land showed later§ that the latter diagram may be applied advantageously in analyzing a plane state of stress at a point.

E. Winkler introduced the influence line in 1867.

Williot proposed, in 1877, his diagram by which the displacements of all points of a truss are obtained as vectors.‖

After Culmann's death, W. Ritter continued his work at the Polytechnikum in Zürich. Ritter's treatise¶ possesses distinction and originality. The

* *Philosophical Magazine*, (4), v. 27, 1864, p. 250.

† See the collection of papers by Mohr, re-edited by himself, "Abhandlungen aus dem Gebiete der technischen Mechanik," Berlin, 1906, Second Edition, 1914.

‡ H. Müller-Breslau, "Beitrag zur Theorie des Fachwerks," *Zeitschrift* des Architekten- und Ingenieur-Vereins zu Hannover, v. 31, 1885, p. 418.

§ *Zeitschrift* des Vereins deutscher Ingenieure, 1895, p. 1551.

‖ Williot, "Notations pratiques sur la statique graphique," Publications scientifiques industrielles, 1877.

¶ "Anwendungen der graphischen Statik nach Professor Dr. C. Culmann," 4 vol. Zürich, 1888–1906.

ellipse of elasticity, proposed and used extensively by W. Ritter, has interested many. No one will deny its value in defining qualitatively the nature of the stiffness of a flexible connection between two elements of a structure.

Some graphical methods refer to continuous beams. Two should be mentioned, one originated by Mohr, the other by Claxton Fidler. Mohr's method is based on properties of the tangents to the elastic curve. Culmann, Winkler, and especially W. Ritter* took up the idea and extended its use. The other method, which has turned out to be simpler, may be interpreted as a graphical solution of the equation of three moments. Claxton Fidler originated it in 1883, Müller-Breslau gave it in an improved form in his "Graphische Statik der Baukonstruktionen", and Ostenfeld developed it further.† L. H. Nishkian and D. B. Steinman, Members, Am. Soc. C. E., have presented the same method.‡

Statically Indeterminate Structures

The theories of statically indeterminate structures may be grouped according to certain leading principles, essentially four.

Maxwell, the great English physicist, originated the first of these in 1864,§ the same year that he invented the stress diagram for trusses. It was on this occasion that he gave the famous theorem of reciprocal deflections which is named after him. In dealing with the statically indeterminate truss he expressed the condition of geometrical coherence by a set of equations in which the redundant stresses are the variables. Mohr, in 1874, derived the same equations by a direct use of the principle of virtual work. The equations are named frequently after Maxwell and Mohr. Mohr's papers on the subject‖ established firmly the applicability of the principle. He included the effects of variations of temperature. Müller-Breslau, in his weighty treatises,¶ included the effects of settlements of the supports and of incorrect initial dimensions of the members, and he extended the method so as to make it apply to solid frames as well as to trusses.

* See v. 3 (published 1900) of his "Graphische Statik," previously mentioned. This volume has the sub-title, "Der kontinuierliche Balken."

† T. Claxton Fidler, *Minutes of Proceedings*, Inst. C. E., v. 74, 1883, p. 196. See, also, his "Practical Treatise on Bridge Construction," Lond., 1887, Fifth Edition, 1924, Chapter 9. H. Müller-Breslau gave the method in a more general form in an article, "Ueber einige Aufgaben der Statik, welche auf Gleichungen der Clapeyronschen Art führen," *Zeitschrift für Bauwesen*, v. 41, 1891, Columns 103–128, and thereafter in his "Graphische Statik der Baukonstruktionen," v. 2, sub-vol. 1, Leipzig, 1892, p. 357 (Fifth Edition, 1922, p. 406); also, v. 2, sub-vol. 2, 1908, p. 37 (Second Edition, 1925, p. 93). A Ostenfeld's treatment of the subject may be found in his "Teknisk Elasticitetslære," Second Edition, Copenhagen, 1905, p. 176; Fourth Edition, 1924, p. 230; in his "Teknisk Statik," v. 2, Copenhagen, 1903, p. 92; Third Edition, 1925, p. 87; and in two papers, "Graphische Behandlung der kontinuierlichen Träger mit festen, elastisch senkbaren oder drehbaren und elastisch senk- und drehbaren Stützen," *Zeitschrift* für Architektur und Ingenieurwesen, v. 51, 1905, Columns 47–66, and "Graphische Behandlung der kontinuierlichen Träger mit elastisch senkbaren Stützen," *loc. cit.*, v. 54, 1908, Columns 57–78.

‡ *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 1.

§ J. Clerk Maxwell, "On the Calculation of the Equilibrium and Stiffness of Frames," *Philosophical Magazine*, (4), v. 27, 1864, pp. 294–299.

‖ See the collection of papers by Otto Mohr," "Abhandlungen aus dem Gebiete der technischen Mechanik," Second Edition, Berlin, 1914, pp. 390–479, especially his bibliographical notes at the end of the paper.

¶ "Die neueren Methoden der Festigkeitslehre und der Statik der Baukonstruktionen," Leipzig, 1886 (Fifth Edition, 1924), and "Die graphische Statik der Baukonstruktionen," v. 2, sub-vol. 1, Leipzig, 1892 (Fifth Edition, 1922).

H. Müller-Breslau (1851-1925), for many years a Professor at the Technische Hochschule, in Berlin, is indeed one of the most distinguished figures in the history of structural statics.* In addition to the many other contributions, he is responsible for the systematizing of what may be called the second leading principle in the analysis of statically indeterminate structures.† The procedure may be looked upon as a variation of that of Maxwell. The essential variables, again, are the redundant stresses (or measures of combined states of stresses, such as bending moments). The linear equations expressing the condition of geometrical coherence are obtained by superposition of displacements due to the individual loads. The coefficients in these equations are displacements due to unit loads. By expressing the individual terms by the principle of virtual work, the equations assume the form obtained by Maxwell and Mohr. The general form, however, in which Müller-Breslau stated these equations leaves the way open to obtain the terms by any method which is available for determining displacements. Müller-Breslau pointed out the usefulness, in this connection, of Maxwell's theorem of reciprocal deflections, and thus he came to the important conclusion that the influence line for a redundant stress or bending moment may be interpreted as a diagram of deflections. During recent years George E. Beggs, M. Am. Soc. C. E., at Princeton University, using this property of the influence line, has developed an experimental method which appears promising in connection with frames which are many times statically indeterminate; he obtains influence lines by observing deflections of models made of paper or celluloid.‡

The third of the leading principles is that of least work. This principle was established brilliantly during the Seventies by the Italian engineer A. Castigliano (1847-84). It should be mentioned that the Italian Count who later became a General, L. F. Menabrea, had stated the principle in 1858.§ The method, however, is named fairly after Castigliano rather than after Menabrea, whose proof was erratic. Castigliano presented his method in a remarkable book, "Théorie de l'équilibre des systèmes élastiques et ses applications" (Torino, 1879). The unknown quantities determined first are the same as those entering in the equations due to Maxwell and Mohr. Müller-Breslau contributed greatly toward making Castigliano's work known; and E. S. Andrews, in England, performed recently a service by translating Castigliano's book into English, under the title "Elastic Stresses in Structures" (London, 1919).

The fourth and last of the leading principles has been used on a number of occasions, but was given a general formulation only recently by A. Ostenfeld, in Denmark. By this principle the unknown quantities upon which the analysis depends primarily, are deformations. The equations of elasticity are of the same form as Maxwell's and Mohr's equations, except that loads and deformations have changed places. Ostenfeld published a book on this sub-

* A list of Müller-Breslau's publications may be found in *Zeitschrift für angewandte Mathematik und Mechanik*, v. 5, 1925, p. 277.

† "Die neueren Methoden der Festigkeitslehre und der Statik der Baukonstruktionen," 1886.

‡ *Proceedings*, Am. Concrete Inst., v. 18, 1922, p. 58.

§ *Comptes Rendus*, Paris Academy, v. 46, 1858, p. 1056.

ject in 1925, entitled "Die Deformationsmethode". Such methods as the slope-deflection method, by which slopes and transverse deflections are the essential unknowns, may be interpreted as examples of application of this general principle.

Among the great textbooks dealing with statically indeterminate structures, with the greatness measured by quality and influence, should be mentioned the following: Müller-Breslau's "Graphische Statik der Baukonstruktionen"; Ostenfeld's "Teknisk Statik" (the last edition of which includes the deformation method, see v. 2, Third Edition, Copenhagen, 1925); and the book by the late J. B. Johnson, the late C. W. Bryan, and F. E. Turneaure, Members, Am. Soc. C. E., entitled "Modern Framed Structures" (Ninth Edition, v. 2, New York, 1911).

Germany, with leaders like Mohr and Müller-Breslau, played an important part in the development of this subject.

Theory of Elasticity

The theory of elasticity has been applied to practical problems in many ways during recent years. A. Föppl (1854-1924), not least through his wonderful book, "Technische Mechanik", the fifth volume of which deals with the theory of elasticity (v. 5, First Edition, Leipzig, 1907), doubtless exerted a great deal of influence in this respect.* In 1920, he and his son, L. Föppl, who later became his successor as Professor at the Technische Hochschule, in Munich, published jointly a two-volume theory of elasticity for engineers, with the peculiar title "Drang und Zwang".

W. Ritz in a notable paper published in 1908† gave emphasis to a principle of minimum which lends itself particularly well to approximate solutions. Admissibility of approximate methods extends the field of accessible problems greatly. Ritz's principle of minimum is based on variation of shape, whereas Castigliano's principle is based on variation of the state of stress. Approximate investigations based on the minimum of energy by variation of shape are described frequently as analysis by Ritz's method.

A. and L. Föppl's "Drang und Zwang" contains many examples of this type of analysis.

Important work has been carried out during recent years on torsion, especially by L. Prandtl and A. Föppl.‡ Prandtl discovered in 1903 the "soap-film analogy", by which problems of torsion may be solved. He showed that the equations governing the state of stress in torsion are the same as those governing certain properties of a soap film deflected by a pressure from one side. Thus, one may transfer solutions by analogy from one field of mechanics to another. The soap film, being easy to visualize and to produce experimentally, and being subject to fairly simple laws of mechanics, has proved to be very useful in this respect.

Much work has been done during the last ten or fifteen years on the subject of the flexure of slabs. Two names should be mentioned: A. Nádai in Germany and B. Galerkin in Russia. Nádai wrote an excellent book, "Die

* A list of A. Föppl's technical and scientific publications, including eighty-five items, may be found in *Zeitschrift für angewandte Mathematik und Mechanik*, v. 4, 1924, p. 530.

† *Crelles Journal*, v. 135, 1908.

‡ See A. and L. Föppl, "Drang und Zwang," v. 2, 1920.

elastischen Platten" (Berlin, 1925), in which many of the recent investigations are described.

H. Reissner, in Berlin, and E. Meissner, in Zürich, have made important contributions to the theory of domes and shells.*

Recent Developments

The plastic stage, occurring when certain stresses exceed the proportional limit or the yield point, has been the object of some theoretical studies during the more recent years.

The action of a column of ductile material presents a problem of this kind. If one wishes to understand this action, it is necessary to consider the changed conditions of stiffness beyond the proportional limit. Engesser and Considère discussed the mechanics of this case some time ago. A fully satisfactory treatment of this subject, accompanied by careful tests, came in 1910 from the hand of Th. v. Kármán, then in Göttingen.† His contribution is one of the most important in the whole vast literature on the subject of columns.

Prandtl and, after him, Hencky have succeeded during the last few years in analyzing the state of stresses and deformations occurring in certain cases of plastic action when one body penetrates into another.‡ Nádai has analyzed cases of torsion beyond the proportional limit.§

Charles Terzaghi, M. Am. Soc. C. E., has attacked the fundamental problems of the mechanics of soils.|| Strikingly interesting is the mutual influence found here of the state of stress and the movement of water. A problem in strength of materials becomes hydraulics.

Finally, may be mentioned an exceptional piece of work, theoretical and experimental, by A. Brandtzæg, not yet published, but presented in a thesis at the University of Illinois, in 1926.¶ The subject is compressive strength of concrete. Following and developing a procedure introduced by R. Böker in 1913,** he applied methods of statistical mechanics to a solid consisting of crystalline parts with planes of weakness distributed by laws of chance in all directions. He succeeded in this way in accounting for a part of the curved portion in the stress-strain diagram, and he explained a number of the phenomena which he found experimentally in cases of three-dimensional states of stress. Here, surely, the subject of strength of materials is in a new phase of development.

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* Concerning this and other modern applications of the theory of elasticity, see the article by L. Föppl, "Neuere Fortschritte der technischen Elastizitätstheorie," *Zeitschrift für angewandte Mathematik und Mechanik*, v. 1, 1921, pp. 466–481.

† *Mitteilungen über Forschungsarbeiten aus dem Gebiete des Ingenieurwesens*, Heft 81, 1910.

‡ L. Prandtl, *Zeitschrift für angewandte Mathematik und Mechanik*, v. 1, 1921, p. 15; H. Hencky, *loc. cit.*, v. 3, 1923, p. 241; L. Prandtl, *loc. cit.*, p. 401, and *Proceedings*, First International Congress for Applied Mechanics, Delft, 1924 (pub. 1925), p. 43.

§ *Zeitschrift für angewandte Mathematik und Mechanik*, v. 3, 1923, p. 442.

|| Series of articles in *Engineering News-Record*, November-December, 1925; also, *Proceedings*, First International Congress for Applied Mechanics, Delft, 1924 (pub. 1925), p. 288.

¶ The theoretical part of this investigation has now been published under the title "Failure of a Material Composed of Non-Isotropic Elements; an Analytical Study with Special Application to Concrete", Det kgl. Norske Videnskabers Selskabs Skrifter, 1927, Nr. 2, Trondhjem, Norway, 1927.

** R. Böker, "Die Mechanik der bleibenden Formänderung in kristallinisch aufgebauten Körpern," *Forschungsarbeiten auf dem Gebiete des Ingenieurwesens*, Heft 175–176, Berlin, 1915; dissertation with the same title, Berlin 1913.

THE PRESENT

It is by no means discouraging to consider the state of affairs in structural mechanics of the present day. Group enterprises of many kinds are promoting research and distributing knowledge. Think, for comparison, of the situation 150 years ago when only a few learned academies performed these functions. Now engineering societies, schools of engineering, and some periodicals do much to make research possible. One periodical of this kind should be mentioned especially. It is the *Zeitschrift für angewandte Mathematik und Mechanik,* published since 1921. Its editor is a man of unusual caliber, Professor R. von Mises, at Berlin. The German periodical, *Der Bauingenieur,* published since 1920, also deserves notice.

In the spring of 1924, largely due to the initiative of Professor C. B. Biezeno, in Delft, a "First International Congress for Applied Mechanics" was held in Delft, Holland. The *Proceedings* published the following year contain much material of practical and scientific value. A Second Congress was held in September, 1926, in Zürich, Switzerland.

A recent enterprise of great value is the "Encyklopädie der mathematischen Wissenschaften", of which Volume IV, which consists of four sub-volumes, has the title "Mechanik". Volume IV has been completed except for a part of Sub-Volume 2. The plan of this encyclopedia is to summarize all the important work of the field.* Another similar enterprise has been started very recently under the title "Handbuch der physikalischen und technischen Mechanik".†

Among the individuals representing structural analysis at the present day, there are some stars of considerable magnitude. Among them four may be mentioned: L. Prandtl and Th. v. Kármán, in Germany; A. Ostenfeld, in Denmark, and S. Timoshenko, formerly in Russia, but since 1922 in the United States. They are men of broad scholarship and creative imagination.

The work of Dr. Timoshenko brings to attention the Russian tradition, of which he is a contemporary representative. One may trace this Russian tradition back to the days of Euler and his influence through the St. Petersburg Academy. Dr. Timoshenko's presence in this country is a reminder, at the same time, of the international character of the science of structural mechanics.

Structural analysis is, of course, intimately allied with and dependent on structural testing. The United States has excelled in the latter field. The grand scale of this experimental work, and the determination to find out, have aroused the admiration of Europeans. In structural engineering this country has also excelled. In structural analysis, it has done well, yet the great ideas have come mainly from Europe.

Hope for future advance in structural analysis lies in the fact that the subject, international as it is, fascinates many in all parts of the world.

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* The article by L. Henneberg, in Sub-Vol. 1, and those by Th. v. Kármán, M. Grüning, and K. Wieghardt, in Sub-Vol. 4, are of especial interest in connection with structural analysis.

† Edited by F. Auerbach and W. Hort and published in Leipzig, Germany.

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PAPERS AND DISCUSSIONS

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IMAGINATION IN CITY PLANNING

By Stephen Child,* M. Am. Soc. C. E.

Synopsis

Usually those who scoff at imagination are the very ones who are the loudest in their condemnation of city planning in general, and of its failure to be constructive—to get results. It is the presence of these individuals in American communities that has, more than all else, prevented progress in community planning; but in spite of them the successes have been notable. If failures have occurred it is because many people have lost contact with the highest ideals. Genuinely constructive city planning cannot be accomplished, in fact, one cannot accomplish much of anything that is worth while, unless one follows an ideal—a vision. It is a truism that has persisted since Bible times that, "Where there is no vision the people perish".

The preparation and execution of city plans will require a certain amount of machinery. A permanent city planning office is best. It should be organized on the simplest lines. Friction and delays may be avoided by some automatic provision for sufficient funds to maintain the office. Existing building codes may need revision and provision should be made for excess condemnation, special assessments, and zoning. The latter is important, but should not be overstressed. The board of appeals is the necessary safety valve.

However, all these are more or less mechanical details. The great need is imagination—vision. City planning problems are best solved by applying principles of art and rules of design to scientific data, because city planning is an art, as well as a science. The kind of knowledge needed is extensive rather than intensive and a degree of detachment on the part of both groups —the city planning commission, and the advisers—is desirable. With the permanent city planning office organized and its duties and purposes outlined, it is advisable, first of all, to prepare a comprehensive civic survey.

Note.—Written discussion on this paper will be closed in **August, 1928.**

* Landscape Archt.; Consultant in City Planning, San Francisco, Calif.

Inevitable growth entails continuous planning. The commission, like the General Staff of an Army, after studying the comprehensive civic survey will prepare its campaign—its program, its city plan. It will be able to demonstrate that planning wisely does not curtail liberty, but rather controls license. It should not fail to secure publicity. It should plan for the old and the new sections of the city. Beware of the "untamed engineer", the city planner of the past; recognize the "feel-of-the-land" principle; and study to understand, perpetuate, and enhance the spirit or genius of the city—its personality. These ideals must all be embodied in the design and become a part of the plan.

City planning is a continuous job that is ended only when the community ceases to grow. Its great purpose is civic orderliness and harmony, and the latter may be considered as the fourth dimension of city planning. These aims cannot be attained without imagination.

This presentation begins, therefore, with an ideal, the result of diligent study of the masters, as well as personal experience, and it will be found to be capable of very practical application.

Ideal City Planning

Home, The Foundation.—Ideal city planning recognizes that the life of the city is the home, which must be wholesome, accessible, and agreeable. If the home is the real heart of the community, there is no gainsaying that every home, even the humblest, must be healthy and accessible with agreeable surroundings. "The city without a slum", therefore, is really no longer an ideal, but a necessity, and all housing must be adequate. Surely this is a great stalk of the tree—perhaps the root itself. Surely, too, it will require imagination, not to mention courage and patience, to abolish the slum, but it must be done.

No doubt there will have to be a certain amount of machinery—of organization—through which it will be necessary for civic imagination to work; and one of the problems will be to keep this machinery simple in character, free from the cumbersome cogs and other gear, frequently involving duplication of endeavor and friction. Then, too, the machine must be well oiled, which means literally, sufficient funds to keep it quietly, smoothly, and continually at work.

First among the tools necessary will be the City Planning Commission and its permanent City Planning Office. It is clear that if annihilation of the slums is proposed cities must have effective building codes. Other necessary tools will be excess condemnation, special assessment, and zoning laws, as well as other legal, engineering, and technical paraphernalia. While city planners, both experts and commissioners, must be sufficiently familiar with these technical elements of the broad subject to recommend and use them intelligently, these are by no means all that is necessary. As John Nolen, M. Am. Soc. C. E., the well-known city planner, stated recently, "There must be engineering plus something, architecture plus something,

landscape architecture plus something". This "plus something" is, in the writer's judgment, imagination and vision.

City planners and all those who are interested in city planning must define more clearly than in the past, the relation of these allied professions to this great problem. The thoughtful are saying with Emerson:

"Do your work. It is necessary to say this often, but nature says it oftener. 'Tis clownish to insist * * * on doing all with one's own hands —but he is to dare to do what he can do best, to do otherwise is to neutralize all those extraordinary talents distributed among men".

The underlying thought behind good city planning is that after the comprehensive civic survey previously alluded to has been prepared, the putting of all this sum of knowledge into the final column of expression becomes an artistic problem, and, for this reason, the same rules which apply to the creation of an artistic design are at the basis of a convenient and workable city—the same proportioning between different parts, the bringing into harmonious relation of the industrial, commercial, or residential quarters, the grouping and linking together of civic and governmental centers, industrial and recreational centers, and the linking of these together with main highways of great width to accommodate the main lines of traffic, with secondary highways for convenience of communication in detailed parts, etc. —all this follows exactly the same rules as govern design.

The need is for a sort of knowledge that is extensive rather than intensive, and there must always be maintained a degree of detachment from the details of the problem, if the real problem itself is to be seen fairly and seen whole. Members of city planning commissions should never forget this. They must give over these details to the various appropriate city departments—to advisers and consultants. Otherwise, they will soon become so immersed in these details that they will lose their sense of proportion, and will not be able to see this great problem fairly and as a whole.

Unfortunately, some have considered civic art as a "'trimming' to be stitched in ever-increasing quantities to the garments of life—they would fill our streets with marble fountains, dot our squares with groups of statuary, twine our lamp posts with wriggling acanthus leaves or dolphins' tails"; whereas the imaginative city planner realizes that in true civic art, real constructive city planning, to quote Mr. F. L. Olmsted, Jr.,

"Regard for beauty must neither follow after regard for the practical end to be obtained, nor precede it, but must inseparably accompany it."

Mention has been made of a City Planning Commission as one of the most important of necessary tools. With park, playground, health, public service, and many more commissions, the question "Why another?" will not be silenced. The answer is: No one of these has any authority beyond its own field and cannot correlate its own problems with any broadly outlined plan for "general welfare". Furthermore, the present tangle of department organization, in the United States at least, encourages jealousy and a pernicious spirit of special preference in absolute disregard of communal interests, and not only that, leaves "twilight zones" where no authority is

operative. A city planning commission animated by the spirit—correlation of all for the good of all—will become the hub of the wheel, enabling the municipal coach to run rhythmically on the broad tires of altruism.

How shall the City Planning Commission and its concomitant, permanent City Planning Office, be organized? The Advisory Committee on City Planning and Zoning, a group of experts appointed by the Secretary of Commerce, has recently, after two years study, recommended the arrangement shown in Fig. 1.

Each appointed member of the Commission serves six years, with the exception that the respective terms of five of the members first appointed shall be for, one, two, three, four, and five years, respectively. There is a provision in the ordinance permitting re-appointment, but each year the Mayor has the opportunity of naming a new member, and, by this means, of infusing new life into the work of the Commission.

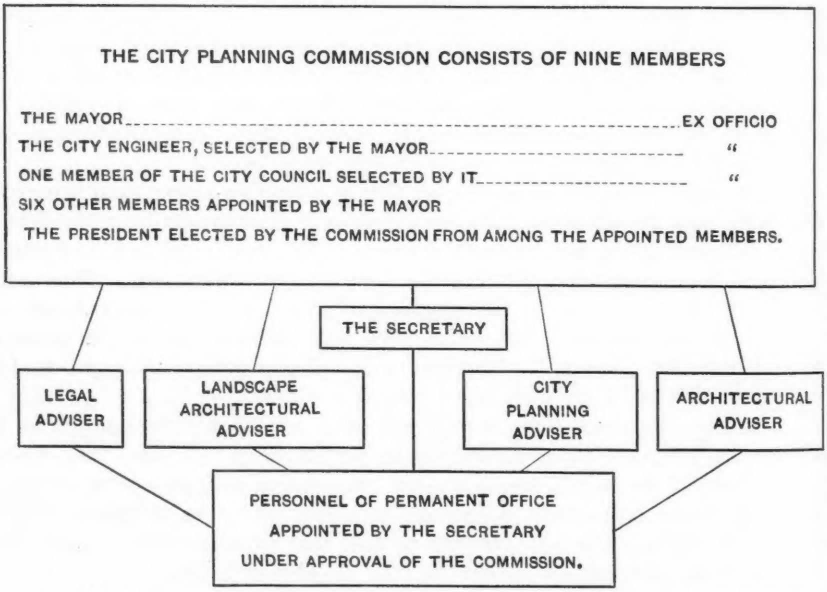

FIG. 1.—ORGANIZATION OF CITY PLANNING COMMISSION AND ITS PERMANENT OFFICE.

Such a City Planning Commission will set up (as Fig. 1 suggests), a permanent City Planning Office, because the work is continuous, not being ended until the community is ready to say that it does not propose to progress or grow any more.

The Advisory Committee prepared a report entitled, "A Standard City Planning Enabling Act," which was published by the U. S. Department of Commerce in February, 1927.* This report is perhaps the most important pronouncement yet published on the subject of city and regional planning.

* This report can be secured by addressing Mr. John M. Greis, Division of Housing and Construction, U. S. Department of Commerce, Washington, D. C.

The City Planning Commission will supplement the imagination and vision of its own members, by the experience, training, and vision of various experts. (See Fig. 1.)

The Survey, A Necessary Preliminary.—One of the first things that the Commission will do will be to prepare a comprehensive civic survey, including a study of all the historical, social, economic, industrial, educational, and engineering problems involved. How can imagination and vision work intelligently, unless there is complete knowledge of what there is to work with? This can only be secured from a comprehensive survey.

The civic survey has been spoken of as "the entrance porch of city planning". If this is so, the porch must be broad and substantial, as becomes a stately home. It will then be the vantage point from which the City Planning Commission and its advisers may gain the broader outlook on their domain. Here will be assembled varied information which is conveniently arranged, kept up to date, criticized, studied, and applied. The heavy responsibility of details and effectiveness will fall on the thoroughly trained and experienced executive officer that the City Planning Commission will find essential to success. Special historical investigations must be made, data in regard to the social, industrial, housing, play, park, educational, and financial conditions of the community must be assembled. All this, in the hands of an intelligent, trained official, will be studied and presented to the Commission, and through it to the public in the form of charts and diagrams.

Poor planning, or the neglect of any planning in the past, will, no doubt, necessitate heavy expenditures for the correction of errors, but this, it must be remembered, "is not a bill for City Planning; it is a bill for surgical service, and the size of the bill cannot affect the need of the operation". If thorough investigation of the City Planning Commission establishes the necessity of an improvement of this character and shows that the money for it will be well spent, financial difficulties due to archaic limitations on borrowing ability or obsolete methods should be thrown into the junk heap. Long-term bonds distributing the heavy burdens of these errors, may be imperative. The City Planning Commission may find it important to investigate thoroughly the financial methods of the city.

Zoning is another important tool for the city planner and may be used according to specific requirements, whether it is for residence, business, or manufacturing, and according to the desirable types of buildings and open spaces on the lots. However, it is very important that these differing regulations be the same for all districts of a similar type. All men must be treated alike. Reasonableness and fairness are the two vital essentials to successful zoning.

Planners should attempt to accomplish only that which is in accord with the law and remember too that, in their efforts to correct existing faults, progress may perhaps come only as the result of a wearing out process.

Comprehensive zoning provides not only for safeguarded residential sections, but for the peculiar needs of industrial districts as well. Manufacturers are thus assured of the great advantages that come from large-scale facilities, undisturbed by the demands of home-owners.

The first thing to be done is to secure the enactment of what is called enabling legislation, that is, State laws that permit cities to enact and enforce zoning regulations. These must be based, in the United States on the so-called "general welfare" clause that occurs in one form or another in the different State Constitutions, that which is sometimes termed "the police power". Twenty-six States have adopted such "enabling" laws. Arizona has now (1927) done this and has done it correctly, after fumbling a little about it two years ago.

It is of the utmost importance too, that such State enabling legislation shall include provision for a Board of Appeals. This has, unfortunately, been omitted from some State zoning laws. Without a Board of Appeals, any zoning law is open to direct attack in the Courts. A Board of Appeals is the necessary safety valve. It must be clearly recognized also, that it is not a legislative body. It is an administrative body, acting under rules prescribed by the Legislature. Its status is the same as a Board of Tax Assessors, or a Public Utility Commission.

City planners must not forget, however, that while zoning is a very important part of city planning, its value should not be exaggerated or over-emphasized, but should always be subordinated to a general program for the city's best good—a comprehensive city plan. A well-considered city plan will include many other phases of the city's life; its major and minor street systems; its homes, parks, schools, libraries, hospitals; its health and transportation interests; all the essential features of a well-equipped city. City planners have sometimes erred in this respect in the United States, for they have undoubtedly over-emphasized the importance of zoning occasionally. It must be admitted that while intelligent zoning has an influence for good on many of the factors of a city plan, particularly of housing and homes, by itself, it is not a universal panacea for all the city's ills. Nevertheless, as a part of a larger program, it has been found to pay the city and its citizens a quicker return than almost any form of civic endeavor.

The Plan, A Problem in Design.—The important result of the efforts of the City Planning Commission, and its advisers is the Master Plan; and the preparation of it demands a thorough understanding of the principles of design. The plan should be a work governed by the laws of unity, variety, harmony, and balance, which are as important in the design of a convenient and beautiful city as in that of a cathedral.

City planning by these and other means affects values and sometimes affects them radically. If it is efficient it will increase them quite generally throughout the city. However, there should be some means of adjusting fairly, those values that the community action of a City Planning Commission may tend to raise or lower. It is capable of the incontrovertible demonstration, however, that except for the "surgical operation" alluded to, comprehensive city planning means spending not more, but more wisely.

The legal aspect of the problem is, indeed, fundamental. Building codes, financial legislation, excess condemnation, assessments, zoning, all demand an able legal adviser, (see Fig. 1) to whom the City Planning Commission may refer all ideas and procedure. His advice will be preventative, as well

as curative, of legal delays and entanglements. Thus advised, the Commission will be able to convince obstructionists that planning a city wisely does not curtail liberty, but rather controls license. Furthermore, the Commission and its advisers must have the imagination to apply the laws of unity, variety, harmony, and balance to designs that permit inevitable growth.

If there is any one thing that transcends another, it is growth. Many have assumed that beautiful pictures and plans accompanied by florid literature, put forth after all too brief a study in the field, will in some mysterious manner solve the question forever, a credulity devoid of common sense. The problem in any large or growing city is not only so vast, but is also changing so constantly that neither expert nor commissioner has the omniscience to solve it in one stroke. New elements such as unheard of means of transportation, for instance, will develop each year. City planners are only now beginning to make adequate provision for the "bird-men" and Zepplinists who are now allying themselves with commerce and transportation, and are demanding landing and starting places—demands that, if adequately met, may very well revolutionize plans.

By all means enlist the range of vision and the varied experiences of the expert, but let it be accompanied by the supreme controlling force, the patient, persistent effort of a permanent City Planning Office. A commission composed of men of vision, tact, and experience, will organize and administer an office that will become a "clearing house" for all civic endeavor. Like the General Staff of an Army, the Commission will prepare its campaign as far in advance as its prescience can foresee, and with continuity of purpose, and more or less constant advice from experts, adapt, revise, and mould the plan in harmony with the unfolding years.

With the complete understanding of conditions and tendencies which the civic survey will give, the City Planning Commission will proceed to define its purpose and program in matured recommendations; will advise, as to the order of new projects, the apportionment of appropriations; but, on the other hand, will refer details of execution to appropriate Departments. Its mere advisory powers, enhanced by tactful initiative, will secure results in various ways: First, perhaps by a closer co-ordination of existing lines of activity; and, second, by improving these. The Commission will employ experts and specialists to design and report on certain portions or details that will fit into the general plan, without undue exaggeration of their specific purpose. For example, it will not advise the expenditure of energies and funds for a civic center or large park, if in its judgment there is more immediate need for the eradication of a slum or for better docks or transit facilities.

The cost of civic improvements is met in various ways. Special assessments and excess condemnation have been mentioned as helpful. As to these, it is believed that generally in developed sections, assessment for benefit is better; while in undeveloped sections, excess condemnation is preferable; frequently a combination of both is advantageous.

As a stimulant to civic ardor, the City Planning Commission should not fail to invite publicity. The public must be systematically informed, from

time to time, of the Commission's purpose, progress, and accomplishment. Wanting the enthusiastic endorsement of public sentiment, "the best laid schemes o' mice and men gang aft a-gley". Notwithstanding its annoyances, particularly in the delays entailed, the hearty support of the average citizen is vital to the success of the program. The "Hausman method" as applied in Paris, which appeals to the aggressive, namely, the forcing of great schemes of civic betterment on an indifferent or unwilling proletariat, is not compatible with democracy and is seldom successful.

It will be the City Planning Commission's responsibility to decide when and where to revise older parts of the city. Splendid civic centers created, or main thoroughfares widened, at great expense, make a spectacular appeal to civic pride, and are often necessary; but equally so, if not more essential, will be the city's new growth. In hilly districts, best suited to homes, the Commission should restrain the thoughtlessness of the past, that permits such atrocities as the further extension of the "checker-board plan". It should work from the outside toward the center, as well as from the center outward, for as with a rapidly revolving fly-wheel, in which centripetal and centrifugal forces must balance, or it flies apart, so is it with city planning, the City Planning Commission being, in very truth, the balance wheel.

In reclaiming the older parts of the city, the besetting difficulties are not insurmountable and must be overcome. Ill-planned, neglected sections, like defectives and cripples, need special treatment, which the City Planning Commission, in collaboration with specialists, will recommend. It will be found that even the desperate, so-called "blighted districts" (potential slums) will respond to forethought and concerted effort. Zoning laws carefully devised and administered will at least prevent the spread of the disease, will isolate and enable its amelioration, if not its cure.

If there is a clear call for a civic center of dignity and beauty, the landscape architect or city planning consultant, in co-operation with architects, will assist in the responsibility of selecting the site, the details of the group of buildings and their surroundings. When necessary he will co-operate in making plans for other groups, school houses, gymnasiums, public baths, branch public library, postal sub-station, police and fire stations, for, with foresight and good design, a number, if not all, of these groups may be effectively assembled about the neighborhood park or recreation center, causing it to become a veritable social center, to the great convenience and uplift of a district. The City Planning Commission and its advisers, in co-operation with the various departments affected, will demonstrate in all such instances that reconstruction is but the renewal of opportunity.

It may be, however, that in the newer parts of the city, imaginative city planning will most effectively prove its worth and here the problems are frequently most urgent, for "past carelessness has queered the pitch", and timely action may restore or secure harmony. "The untamed engineer", yesterday's city planner (and he is by no means extinct), cares only for straight lines and grades, up and down at uniform rates. Uniformity is his creed; sidewalks must be just so many inches from the lot line and above the gutter;

trees must be always outside the walk and an equal distance from the curb; streets must be built in straight lines without regard for topography. He is utterly oblivious to the beauty of a curving line, or to views and vistas that might have been, but, alas, will never be. Everything must fit into mechanical conformity. The "untamed engineer" attacks his urban problem with violence—the artist engineer, the imaginative city planner, adapts himself to Nature sympathetically.

The old Gothic cities, like Nuremberg and Rothenburg, had their marvelous "feel of the land", the intimate and sympathetic adjustment of the lines and grades of streets, as well as of the character and quality of buildings, to the beauties of natural contours, and to the inspiring appearance of some centrally located cathedral. There is little of this in American cities. These thoughts suggest another thought of equal import. Not only the inherent physical characteristics, but the spirit or genius of the city, so closely interwoven, must be appreciated, enhanced, and sustained. The cities of Nuremberg and Rothenburg, on the Continent, and Oxford and Stratford, in England, have accomplished this under the guidance of men trained to the appreciation of esthetic values, men who were the equivalent or the forebears of the landscape architects, architects, and city planners of to-day.

These are the modern city planners' ideals. Follow this gleam and your city will become one "that is at unity with itself".

The creation, adoption, application, development, and revision of a comprehensive, tentative plan constitutes an imposing program, one that will demand insistence, persistence, consistency, patience, and publicity, one that will make insistent demand upon imagination, optimism, and tenacity of purpose.

The Future Provides for Growth.—The future city, a living organism, guided by a definite art purpose, includes inevitable growth in its ideal. It is a conviction among the more thoughtful and progressive city planners, that it is only a matter of time until a city plan office, such as is herewith suggested, will, in a comprehensive and constructive manner, initiate and control, either directly or indirectly, all city plans. Unquestionably, some such method must take the place of the prevailing slip-shod lack of purpose; there must be substituted "order in place of chaos in town growth—the wisdom of prevention as compared with the wastefulness of cure".

Leaders in the city planning movement, not only in America, but in Europe, are realizing that cities need to cease planning only when they cease to grow; that every community that hopes to achieve results (physical as well as economic) from its city planning efforts, must not only consider it as a continuous process, but realize that this continuity of endeavor must follow a carefully prepared program.

Furthermore, it is also realized that the day is now passed, when city planning service can be completely satisfied by a two or three weeks' visit of an expert, with intensive but necessarily hurried study followed by a report. While documents thus prepared often contain valuable advice and suggestions, they are usually laid aside and soon forgotten. Their recommen-

dations and suggestions must be followed up, and amended to suit changes due to inevitable growth. This being the case, it is now realized that constant civic growth needs the more or less regular and continuous guidance of a trained city planning consultant.

Every prosperous growing community needs to prepare such a city planning program—one that will recognize the many questions in regard to city planning that are being discussed in each community, as well as new ones that will be constantly arising. Perhaps the assertion may be pardoned, that this means much more than zoning or the widening of important thoroughfares, although these would undoubtedly be important factors in such a program.

Continuous city planning means correlating all such items as these and more, into a city planning policy or program that will control the city's growth, and while directing it along lines that are most appropriate, also lead it away from inharmonious projects—a program, however, that will be flexible enough to permit of adjustment to new conditions, particularly to that of inevitable growth. All such adjustments should be made with care and under competent advice.

There is also general agreement among city planners that the first step is the preparation of a comprehensive civic survey, to which allusion has already been made.

It is in the preparation of such a civic survey and city planning policy, or program, and in the regular guidance of the community's city planning efforts, that the city planning consultant can be most helpful. Instead of the two or three weeks' visit and intensive study, followed by an elaborate and almost useless report (a common method of the past), the consultant should be present at one or more of the regular monthly meetings of the City Planning Commission. He should consult with the Commission and answer all questions frankly. Should matters arise that need more thought or detailed study, he should give to such problems the necessary effort, and report either in writing, or orally, as soon as possible. He should direct the civic survey mentioned, and any field investigations, or the preparation of any special maps or drawings, that might become necessary. The actual field work of such investigations, and the drafting of such plans, should be done, however, at the city's expense (probably in the City Engineering Department) and be subject to no charges on the part of the consultant. His services should be strictly those of adviser on plans and program, and his fee a nominal monthly or quarterly retainer.

As the civic survey progresses a start could be made on a preliminary zoning study and its plans. The consultant should direct these special studies and their necessary office and field work, which would probably be done, as mentioned, in the City Engineering Department. It is not unlikely that such a program would call for at least one new assistant in the city engineer's office—a city planning assistant, one with special training in such matters, probably a graduate of one of the city planning schools. It is clear that this method centralizes the work and locates all records pertaining to it in the

City Engineer's office, where they will be available at all future times, and not (as so often occurred in the past) isolated in the office of the consultant, perhaps many miles away. There are many other advantages to some of which allusion has been made elsewhere by the writer.*

Conclusion

In conclusion, it may be stated that the great purpose of city planning, civic orderliness—civic harmony—cannot be attained without imagination. Harmony has been recently spoken of as the fourth dimension of city planning, an interesting thought, stimulating to the imagination. The City Planning Commission should be in very truth, "The Official Imagination of the City".

* "City Planning Procedure," *City Planning Quarterly*, October, 1927.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

QUALIFICATIONS OF CONTRACTORS ON PUBLIC WORKS*

By Frank T. Sheets,† M. Am. Soc. C. E.

Synopsis

In this paper, the writer has attempted to diagnose some of the ailments of the construction industry as related to public works. Practical methods of gauging the responsibility of contractors are described, and the problems confronting the public official are analyzed. Bad practices of the several groups comprising the construction field are disclosed and remedies suggested. Responsible contractors are urged to make a united effort to better conditions, and related industries, as well as public officials, are urged to cooperate.

Introduction

The subject of qualifications of contractors on public works may be approached from many angles. The detailed analyses of equipment, organization, and finances required for various types of construction projects are properly related to this problem. However, for the sake of brevity, this paper will be confined to the determination of responsibility of contractors engaged in public works contracts.

A survey of such contracts executed throughout the United States during the past decade will show a long procession of disastrous failures. These failures have not only resulted in great financial loss to the contractors and their bondsmen, but have cost the public millions of dollars in inconvenience and delays. Consequently, the protection of the public from the irresponsible contractor has become a problem of great importance.

Note.—Discussion on this paper will be closed in **August, 1928.**

* Presented at the meeting of the Highway Division, Denver, Colo., July 14, 1927.

† Chf. Highway Engr., State Div. of Highways, Springfield, Ill.

Causes and Results of Failure of Contractors

The inelasticity of legal requirements with which public works contracts are surrounded renders it impossible for contracting parties to make proper adjustments to meet sudden changes in economic conditions, transportation facilities, or physical situations. Consequently, there are occasional failures of absolutely reliable, experienced, and financially capable contractors. However, the majority of failures are directly traceable to inexperience, lack of funds, and lack of proper equipment.

The novice in the construction industry who sees visions of enormous profits in public works contracts, seems to have no difficulty in securing sufficient financial backing to make the first plunge. Many contracting organizations are composed of a construction novice backed by partners or stockholders who have funds but know nothing about contracting. These inexperienced organizations frequently bid recklessly, and have no difficulty in securing contracts in competition with experienced contractors whose bids reflect years of accurate cost accounting on contracts performed. The novice and his backers, like the reckless plunger, must be saved from themselves; and the public must be saved from them.

In many cases, irresponsible bidders have received contracts at figures slightly lower than those of responsible bidders. The resulting inconvenience and delay caused by failure have cost the public many times the difference in the bids.

Bad Practices in Allied Fields

The great increase in the volume of public works contracts in recent years has rendered it absolutely imperative that standards be set up for gauging the responsibility of contractors. It demands many changes in the business policies of the various groups composing the construction industry.

It has been the custom for the surety bond companies, the material producers, the bankers, and the contractors themselves, to denounce vigorously the lack of vision of public officials in awarding contracts to irresponsible parties. At the same time, each of these agencies has perpetuated practices and policies which have made the path of the novice, the crook, and the plunger one of roses, and which have forced the public official to "stand the gaff" of public criticism and even of suspicion of his integrity if he were to carry out the dictates of his unbiased judgment in the elimination of irresponsible bidders.

Gauges of Responsibility

The Illinois Department of Public Works and Buildings, Division of Highways, has faced this problem and has tried to meet it. This paper will indicate some of the experiences and a few of the results.

The laws of Illinois require that contracts be let to the lowest responsible bidder. The Division of Highways realized fully that no arbitrary determination of the responsibility of a bidder could be made without suspicion of its motives. Therefore, some years ago, it adopted a form of financial statement and experience and equipment questionnaire which each bidder was required

to submit with his proposal. It established as a result of its own judgment and the judgment of a large number of reliable contractors, the minimum financial qualifications required for the execution of contracts of various sizes; it determined from years of experience the minimum requirements of equipment for various sizes of contracts; it analyzed carefully the pre-arranged plans of contractors for handling every project; and with all these data for its guidance it determined the responsiblity of bidders.

The usual practice is to require that a contractor for a paving or bridge project have available in net quick assets approximately 20% of the contract price. In some instances where a contractor has completed successfully many projects previously, it has been possible to reduce this percentage to some extent. In other cases, where the work was extremely hazardous, even a higher percentage of net quick assets has been deemed advisable.

Sometimes the judgment has been erroneous, and an apparently responsible bidder has failed during the test of construction, but considering the fact that the State has carried on $150 000 000 worth of road and bridge contracts during the past few years under this policy, the number of failures is gratifyingly small.

Treatment of Irresponsible Bidders

After determining that a bidder is irresponsible, a letter is sent, rejecting his bid, and giving a complete recitation of the facts upon which this judgment is based, these facts, of course, being taken from the sworn statement filed with the bidder's proposal. Should the bidder resent the treatment accorded him and endeavor to place the Department officials in an unfavorable light, they can then publish this letter and vindicate themselves from any charges of favoritism or attempted juggling of contracts.

In many instances when bids have been rejected, it has been suggested that, although the bidder's qualifications were not sufficient to justify awarding him the specific contract under consideration, further bids on smaller and less hazardous work would be welcome. It has been gratifying to see many such contractors take work for which they were qualified and thus develop gradually to the point where they were able to undertake almost any project.

Standard Questionnaires

The questionnaires now in use are an enlargement and refinement of the original forms. They are practically identical with the questionnaires adopted by the Joint Conference on Construction Practices,* in which the Society in common with various other interested bodies had a part.

The development of these questionnaires by joint committees of public officials, contractors, bond writers, bankers, and other members of the highway industry has been a distinct contribution to the construction industry. The writer recommends them most heartily to every engineer and public official. Illinois authorities have absolutely proved their value and would not think of carrying on work without them. It is interesting to note that in awarding the large volume of contracts previously mentioned, not a single suspicion of favoritism nor charge of wrong motive has arisen.

* *Proceedings*, Am. Soc. C. E., March, 1926, Society Affairs, p. 259, *et seq.*

Lack of Support for Public Officials

Nevertheless, the use of this questionnaire has caused some embarrassment because the Department has had very little co-operation from the highway industry at large in its efforts to solve this problem in its State work. Many times, when the questionnaires have shown bidders to be absolutely unqualified for the work they were seeking, reputable surety companies have stated their willingness to write bonds on these contracts, and, in some instances, these companies have urged the State to make such an award.

In one instance, the financial statement showed the bidder to be very weak. When the contractor learned that the engineers were considering the rejection of his bid, he made an impassioned appeal for them to give him the contract. Most of the property of his aged father had been involved in past contracting misfortunes, and this one job, if awarded, would enable him to retrieve his father's fortune. A reputable surety company urged the authorities to make the award and stated that it would personally see that the work was completed in proper form. Sentiment got the better of judgment and the award was made. The work dragged along in a dilatory fashion, the Department was bitterly censured by the people in the community involved, and the industry as a whole has not been helped.

Formerly, the submission of bidders' bonds instead of certified checks with proposals was permitted at road lettings. Irresponsible bidders seemed to have no trouble in securing bidders' bonds. This has led to the discontinuance of such bonds and now all proposals must be accompanied by certified checks amounting to 10% of the bid. Even under this provision, irresponsible bidders have been able to secure certified checks, and it is quite certain that they have been helped in this by agents or brokers of surety bonds.

Apparently, the banks have been entirely too liberal in extending credit to irresponsible contractors. In the face of financial statements that are very weak, bidders seem to be able to get letters from banks guaranteeing the extension of a liberal amount of credit. Certainly no banker should want to risk making loans of this character when the financial condition of the contractor is such that a public official, who has nothing to lose personally, hesitates to make an award of the contract.

In one instance, liberal credit was extended to a contractor with the understanding that he would assign payment estimates to the bank. The bank neglected to file the assignment with the State Department. After all payments on the contract had been made and the contract was closed, and after the contractor had failed to meet his obligations with the bank, the assignment was presented for the Department's consideration. The result was that the banker gained some valuable experience in how to "hold the bag". Banking institutions should be much more conservative in extending credit to the contracting industry, especially on hazardous construction.

It seems strange, also, that manufacturers of equipment and producers of materials entering into contract work should be willing to extend the same business courtesy and the same credit to the "would be" contractor that

they do to reputable firms with millions of dollars of successfully completed work behind them and plenty of cash in the bank.

In order to minimize the interest charges necessary to complete State road contracts, Illinois has allowed payment estimates on materials delivered on the ground but not incorporated in the work. This policy was adopted on the recommendations of some of the most responsible contractors. Yet, this practice has to some degree injured responsible firms, because some contractors who are very weak financially have been able to buy materials without making assignments of these estimates and have then been able to use these estimates for current funds to finance labor and machinery payments while the material companies blandly waited for the cash.

The machinery companies have also been most sympathetic and brotherly in their treatment of the irresponsible contractor. In one instance, the machinery was delivered on the job on a time-payment basis. The contractor then went to the bank and borrowed money with which to run the job and put up the machinery as collateral to secure the funds. For ingenuity in financing, that is almost unexcelled.

Responsible Contractors

Enough of the activities of the irresponsible bidder and his associates. The responsible contractor is equally guilty in many instances. He goes right ahead buying his surety bonds, buying his equipment, and buying his materials from the very parties who contribute to his unfair competition by extending unlimited business courtesies and credit to irresponsible firms. Then he consoles himself by attending the annual meeting of contractors and singing a long lament about the deplorable conditions surrounding the contracting business. He complains to the engineer and public official about these conditions and expects them to solve the problem, whereas he should, by concerted action, demand proper co-operation from his business associates.

The Illinois Division of Highways has insisted on absolutely fair prices for road and bridge work. It does not want contractors to do road work in Illinois and lose money, neither does it want them to make more than a fair profit. It has insisted on clean, keen, and open competition and its bids have proved that this exists. However, some of the most experienced and strongest contractors have taken work at figures which are most gratifying to the State, but which cause the Division to look with admiration and even wonder at their estimating ability. It has been somewhat astonished to receive the explanation that the work was taken in order to keep their equipment busy. Why fuss at public officials about the low prices prevailing in public work when the most responsible men in the business repeatedly establish such prices? Either the prices are right and the lament is wholesome propaganda, or the contracting industry needs to clean its own house.

Conclusion

In this paper no attempt has been made to set up definite numerical limits, nor mathematical formulas for establishing the responsibility of contractors. The essentials of honesty, ability, experience, and other elements

which go to make up a successful contractor have not been discussed. Probably, the subject has been treated from the prejudiced viewpoint of the public official battling wearily with a seemingly endless problem; certain horrible examples may have been cited which are the exception rather than the rule; but if this paper has resulted in pricking the conscience of the banker, the machinery manufacturer, the material producer, the bond writer, and the responsible contractor; if it has given a warning to the embryo contractor that he must learn to walk before he can hope to run; if it has been able to point out the challenge that this problem gives to the Engineering Profession, then it will have served its purposes.

It is urged that engineers and public officials who are struggling with the problem of awarding public works contracts, give most earnest consideration to their moral obligation to eliminate, in so far as their legal rights will permit, the irresponsible bidder. They should use sworn financial statements, and equipment and experience questionnaires as tools in performing this work in such a way that no possible reflection can be made on their sincerity of purpose nor honesty of motive. In urging this course of action, the writer has only the interest of the general public in mind, and would not even wish to infer that any collusion between public officials and contractors should be encouraged, because nothing could be more despicable nor less in keeping with professional ethics or good business conduct.

It is a matter of satisfaction that conditions are no worse. They can be made much better, and it is the duty of all elements of the construction industry to proceed sanely, rationally, and intelligently to a solution of this most important and perplexing problem.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

LOAD DISTRIBUTION IN HIGH ARCH DAMS

By R. A. Sutherland,* Esq.

Synopsis

Every year the construction of arch dams involves a great expenditure of money, but methods of design generally adopted in the past have not made full use of the strength of the materials involved. Any method of design that will enable the material to be utilized more effectively, therefore, will effect large savings and increase the confidence placed in this type of structure. A prerequisite to successful design is an exact knowledge of the various forces acting on and in the dam, and the present contribution discusses the principles and methods by which such knowledge may be obtained.

On account of the complexity of the factors involved, previous methods of load analysis were based on many assumptions which aimed at reducing these factors to a form susceptible to mathematical treatment. The principal assumptions are herein reviewed, particularly those relating to the so-called cantilever action. The assumption most strongly attacked is that which regards the arch as being uniformly loaded, and a powerful method of correctly determining the actual non-uniform beam and arch loading is explained.

The methods proposed are based primarily on beam and arch deflections and by means of charts and simplified procedure the application of these methods is made comparatively simple and accurate, and dependent on the minimum number of arbitrary assumptions. When the actual loading on beams and arches under any given set of conditions is known, the resultant stresses may readily be determined, and such modifications in design may be made as are required to enable the dam to carry the load most effectively. The lines along which improvements in design may be made, are then suggested.

A satisfactory method of arch dam design requires a more complete knowledge of actual temperature variations in concrete, of its elastic properties,

Note.—Written discussion on this paper will be closed in **August, 1928**.

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and of the effect on the dam of the elastic properties of the foundation rock. Such knowledge will be obtained mainly from measurements and observations on existing dams. Apart from the incompletely known effect of these factors, arbitrary assumptions in the past have led to errors and should be avoided.

Load Distributions

A number of published investigations on this subject have appeared during the last few years (see Bibliography). Of these, some are so highly mathematical as to be beyond the scope of many engineers, whereas others, in order to obtain simplicity, have introduced assumptions which militate against their accuracy. It is the writer's hope that his method may indicate, perhaps, the lines on which a convenient and accurate analysis may be evolved. To avoid covering elementary ground, a general acquaintance with recently published investigations is assumed, in particular with that* of Fred A. Noetzli, M. Am. Soc. C. E. The latter is a very clear statement of the whole problem, and includes comparatively simple methods for the analysis of load distribution which are extremely useful in the preliminary design of an arch dam. To promote uniformity Mr. Noetzli's symbols have been used as much as possible. It may be mentioned that the work of Professor Guidi, of Turin, Italy, is of great value to any engineer interested in arch analysis, and should be better known in other countries. Guidi's analyses, although developed primarily from an academic standpoint, are direct and may readily be adapted for practical use, as is shown by their extensive adoption in his own country. For instance, his methods were used in the design of such important multiple-arch dams as that at Tirso, which is probably the largest of its kind in the world.

The arch dam may be regarded as a curved plate of varying thickness and generally of irregular outline, restrained only along a part of its periphery, and subjected to non-uniform loads due to water pressure, temperature, and concrete shrinkage. The single feature favorable to a stress analysis, as against the case of structures, such as bridges or buildings, is that the total water load is known with absolute accuracy. This load, however, is not always the most important one in producing stresses, and certain assumptions have to be made in regard to temperature and shrinkage loads.

Most arch dams have been designed on the basis of the "cylinder formula", in which it is assumed that the dam is composed of a number of horizontal arch slices, each consisting of a part of the wall of a complete thin cylinder. It has always been recognized that this assumption is far from representing actual conditions, and to compensate for it, the stresses allowed in design are generally lower than in other types of structures using the same materials. In time, this tacit confession of ignorance must be replaced by more complete and more exact knowledge of the distribution of load in an arch dam, and it seems reasonable to expect that such knowledge will permit the use of higher stresses and enable modifications in dimensions to be made, which will result in economy of material and, at the same time, in a greater degree of confidence in the safety of such structures.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

Before proceeding to the examination of actual stress distribution in an arch dam, it will be well to examine how far the cylinder theory may be assumed to hold good, even considering that the dam may be divided legitimately into a number of independent arch slices. The equation:

$$t = \frac{P_e\, r_u}{f} \qquad (1)$$

is based on the assumption of a perfect and complete cylinder subjected to uniform pressure, the thickness of which is negligible in comparison with the radius. Errors are obviously introduced in the following respects:

(1) The cylinder may not be perfectly circular. A flattened cylinder will obviously endure greater stresses because of bending moments in the planes of the major and minor axes. Arched dams are bound to vary slightly from a true circular shape, but errors due to this cause are probably not great, and of course will be introduced by any method of design.

(2) The ratio of thickness to mean radius in an arch may be as much as 0.72 (Shoshone). It is small only in the upper portion of a dam where direct arch load is small. A high ratio does not, of course, effect the average stress on any section but causes an inequality between the stresses on the intrados and the extrados.

(3) Probably the greatest error is introduced by the fact that, on account of restraint at the sides of the gorge, all parts of the arch ring are not equally free to deflect radially, as they would be in a complete circular ring. This restraint at once introduces bending moments which have the effect, not only of altering the distribution of stress at any given section, but also of producing a variation between the average stresses at different sections. The analysis of a fixed ended arch under uniform load is given in Appendix I, and, by means of Figs. 11 to 13, the actual stresses at four points in the arch may be found at once for any given conditions. These points are the extradosal and intradosal points of the crown and abutment sections, respectively. It should be mentioned that in this discussion the arch is assumed to be fixed at the abutments. This fixation, although perhaps not absolutely in accord with actual conditions, is considered to be more correct than Mr. Noetzli's assumption of a two-hinged arch, for example, which is inconsistent with the assumption of a fixed ended cantilever.

It will be evident from the foregoing that the cylinder formula, except for small thin arch dams, is unworthy of the confidence frequently placed in it. Most attempts to evolve an alternative basis of design have been along the lines of complicated mathematical theory in which, in many cases, the dam was assumed to be confined between vertical abutments and to have a horizontal base. William P. Creager, M. Am. Soc. C. E., expresses this opinion clearly,* stating further that various compensating and cumulative effects, such as stresses due to temperature changes, moisture content, etc., "are sufficient to nullify practically the basic assumptions of such investigations." To the writer, however, the fact that certain assumptions must be made with regard to temperature, shrinkage, etc., appears to be no warrant for

* "Masonry Dams," N. Y., John Wiley & Sons, 1917, p. 154.

evading the investigation of the loads caused in a dam in the fulfillment of its primary purpose, namely that of retaining water. The load due to the water is known with absolute accuracy.

The object of the present paper is to make some small contribution toward:

(*a*) The determination of the distribution of the primary or water load, and of the resultant stresses.

(*b*) The determination, with suitable assumptions, of the effect caused by secondary loads due to temperature, shrinkage, etc.

(*c*) The indication of possible variations in design and construction with a view to a better utilization of material and consequent economy.

The fundamental distribution of load in an arch dam has been stated with great clearness in most of the papers mentioned in the Bibliography, and will be clear from the following considerations. An arch can take its full load only if it is perfectly free to deflect. The crest sections of an arch dam approximate this condition of freedom, but at points nearer the base a greater restraint comes into play, until at the base itself no deflection is possible, and this has led to the conception of the "vertical cantilever"; that is, the dam is assumed to be composed of a series of imaginary vertical strips,* each resting on a series of imaginary horizontal arch strips, constituted of the same material. Among the arch strips, the degree of freedom is a maximum at the crest and decreases to zero at the base. The result is, obviously, that the lower arches take less and less of the total load, which is progressively transferred to, and taken by, the cantilever strips. Assuming the cantilevers to be fixed at the base, a considerable bending moment will be developed there. If on the other hand the cantilevers (which now become beams) are assumed to be hinged or freely supported at the base, no bending moment can be developed there, and the effect of the beams is then merely to transfer load from the more heavily loaded arch strips to those less heavily loaded. Bending moments will occur in the beam itself, but not at the base. Both these cases are dealt† with very clearly by the late R. Shirreffs, M. Am. Soc. C. E., on the basis, however, of assumptions "some of which", Creager states,‡ "are impossible, and others of which are unusual".

It will be well at the outset to examine the legitimacy of the various assumptions made by previous writers. The principal of these are:

1.—The site section is of a rectangular or other regular shape.
2.—The up-stream and the down-stream faces in a vertical section are rectilinear.
3.—The geometrical shape of every vertical section is similar.
4.—The horizontal cross-section of the vertical cantilever is rectangular.
5.—The load distribution on any horizontal arch slice is uniform.
6.—Shear deflection may be neglected.
7.—Young's modulus is constant.

Assumption 1.—Practically all authorities assume vertical gorge sides and horizontal floor, which typifies the average dam site very remotely. For

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* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 62, Fig. 1.
† *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 155.
‡ "Masonry Dams," p. 154.

example, Noetzli dismisses variations from this assumed site section by stating that:* "In general it will be desirable to investigate also the sections at about the quarter-points of the highest arches". The writer wishes to emphasize, not only the desirability of investigating the sections at about the quarter-points of the highest arches, but the absolute necessity of doing so if a correct load distribution is to be obtained.

Cross-sections of the sites of several of the largest dams in the world are shown in Fig. 1. The approximate dimensions are also given. The Shoshone and the Pathfinder Dams are the only ones which approach a rectangular shape. The error of the assumption in the majority of cases is very evident.

However, the dimensions of most masonry dam sites can be reduced to a mathematical formula of the type, $x = Ay^m$. This fact makes it possible to obtain simple expressions for the quantity of masonry in any type of dam built in such a site.

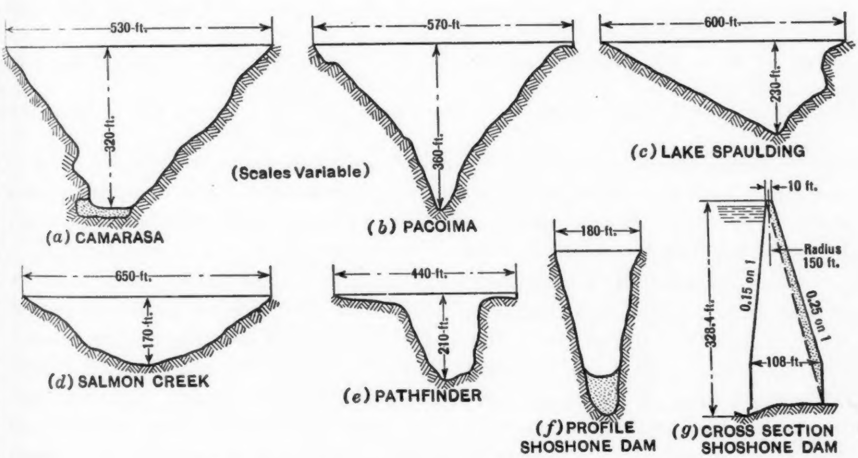

FIG. 1.—SITE SECTIONS OF VARIOUS DAMS.

Assumption 2.—Most writers consider the dam to be of triangular section, which is the theoretical shape for both gravity and arched dams. Many small dams, such as the Barren Jack Creek Dam, in New South Wales, and Las Vegas, in New Mexico, are practically triangular in section with a slight thickening at the crest. In most cases, however, the section is trapezoidal, as shown in Fig. 1 (g). The extra material more than the theoretical amount is shown stippled, and although this may appear an inconsiderable addition it represents, toward the crest, a relatively large increase on the theoretical thickness, and causes a considerable divergence from the results evolved on the triangular profile. Noetzli finds the deflection for a trapezoidal cantilever by first computing it for a triangular and for a parallel-sided section, and taking a proportionate figure between the two. This also introduces errors. Fig. 2 shows the deflection at any point of a trapezoidal cantilever, supporting a water load,

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 25.

for various values of the ratio, $\frac{\text{crest thickness}}{\text{base thickness}}$. The error of assuming a linear relationship between this ratio and crest deflection is evident.

In the case of the constant-angle dam, the section is not even trapezoidal. Shirreffs* has indicated how this variation from the assumed section can be allowed for, but his complicated equations are of little use in any practical case. The arithmetical method explained by the writer covers any actual case that may arise in practice.

Assumption 3.—In the case of dams of homocentric curvature, any cross-section is, of course, similar in shape. In constant angle dams, however, the section varies according to its position. The method of analysis, to be correct, should take cognizance of this fact.

Assumption 4.—The magnitude of errors introduced by the assumption that horizontal cross-sections of the cantilever are rectangular, becomes greater as the ratio, $\frac{\text{thickness}}{\text{radius}}$, increases. Fig. 3 gives the value of the coefficient of t^3 to be used in finding the moment of inertia of radial sections, $I = C_I\, t^3$, for various values of the ratio, $\frac{\text{thicknes}}{\text{up-stream radius}}$. It is seen that errors under this head become important only for arches which are thick as compared with the radius, but it must be remembered that in computing beam deflections errors are cumulative. For preliminary work, this correction may be neglected, but for accurate work the correct values of I should be used.

Assumption 5.—Most authorities have tacitly assumed that the distribution of load on any given arch slice is uniform, or at least that variations therefrom make no essential difference to the arch deflection. The error of this assumption was first pointed out by Guidi, and is such that, in many cases, it stultifies the results of elaborate calculations. Curiously enough, Guidi, reasoning on analogy with the case of a restrained rectangular plate, assumes the flanks of the arch to take a greater share of load than the crown. It appears probable that the reverse is the case.

A small variation from uniform distribution of load on an arch will cause a difference in crown deflection that is out of all proportion to the magnitude of such variation. Non-uniform arch load may best be taken account of by the use of influence curves. (See Appendix II.)

Assumption 6.—The assumption has generally been made that shear deflection is negligible. As every engineering student knows, shear deflection is important only in beams or arches the thickness of which is great compared with their length. It may sometimes be safely neglected for the central cantilever sections of an arch dam, or for the highest arch slices, but considering the ease with which it may be computed, it is believed to be desirable in all cases to allow for it. The neglect of shear deflection will generally cause an error of from 5 to 15 per cent.

Assumption 7.—Much discussion has taken place as to the exact value to be assigned to Young's modulus for concrete. In the design of a dam, assum-

* *Transactions*, Am. Soc. C. E., Vol. LIII (December, 1904), p. 155.

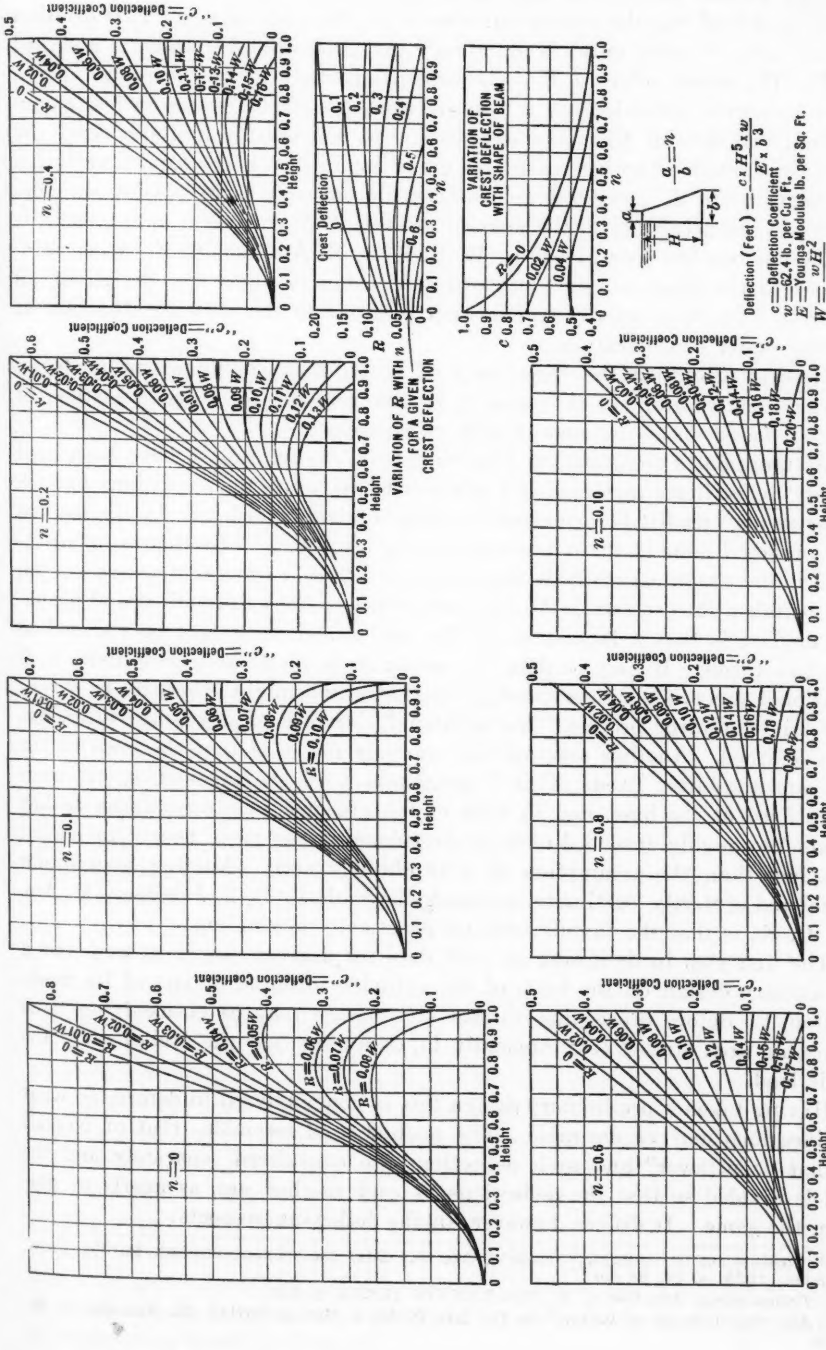

Fig. 2.—Cantilever Deflection Curves.

ing the method used to be approximately correct, the determination of the division of load and the consequent stresses, is the main object. This division of load (due to water only) is practically the same whatever value is assigned to E. The exact value of E is, however, important in two respects: First, when measured deflections on a dam are used as a check on theoretical calculations; and, second, when the stresses due to temperature and shrinkage are to be considered. Another feature of great importance is the known variation of the value of E, according to whether the concrete is in tension or compression, and according to the amount of stress. A great deal of information on these points has been given* by W. K. Hatt, M. Am. Soc. C. E. Uncertainties as to the exact physical properties of concrete affect the design of all concrete structures equally and are not peculiar to the class of structure at present under consideration.

Having examined the legitimacy of seven main assumptions made by previous contributors to the subject, it now remains to devise a method of analysis based on as few assumptions as possible.

Assumptions 1 to 5, and, to a lesser extent, Assumption 6, have been used mainly to facilitate mathematical analysis by reducing the conditions to such forms as may readily be expressed by mathematical equations. In the method now proposed none of these assumptions has been made. By the use of arithmetical and graphical methods, the exact conditions, as far as they are known, may be taken into account. Another advantage of the arithmetical and graphical method is that a large part of the mechanical work may be reduced to standard forms. It may be done by one or more intelligent computers, with occasional supervision, thus speeding up that which might otherwise be a very slow trial-and-error process. Assumption 7, for lack of anything better, has been retained. Another assumption, expressly made, is that the ends of the arch are absolutely fixed. This is consistent with the assumption of cantilevers fixed at the base, and in view of the precautions always taken to key a dam thoroughly into bed-rock, is considered to be more nearly in accord with fact than the assumption of a two-hinged arch. Another assumption considered perfectly legitimate, although disputed† by B. F. Jakobsen, M. Am. Soc. C. E., is that the foundations are practically unyielding.

The first step in designing an arch dam for a given site is to work out a preliminary design on the basis of the cylinder formula, or one of its modifications.‡ Before proceeding further this design may be checked for "true maximum arch stress" (see Appendix I), assuming that the arches carry the whole load.

Having taken a preliminary design this is now analyzed to determine what modifications may be desirable. The method used resembles that of Noetzli in that "cantilever" and arch deflections are considered separately and the load is divided so that the deflections by each method are, as nearly as possible, the same. It differs, however, in the following respects:

* "Researches in Concrete," *Bulletin No. 24*, Eng. Experiment Station, Purdue Univ., November, 1925, p. 14, *et seq.*

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 102.

‡ See "The Control of Water," by the late Philip A Morley Parker, M. Am. Soc. C. E., p. 403.

(1) Vertical sections are taken at several points. Generally, sections at every 10° will be sufficient, the angles being measured at the center of curvature of the crest. In the case of constant-angle dams this point differs from the center of curvature of arch slices at other levels. With the latter type of dam also, the flank sections may approach or even exceed a gravity section.

(2) The method of determining cantilever deflections is arithmetical, and involves as a preliminary step, the determination of the bending moments at various levels. These are useful in determining beam stresses. Shear deflection is considered separately.

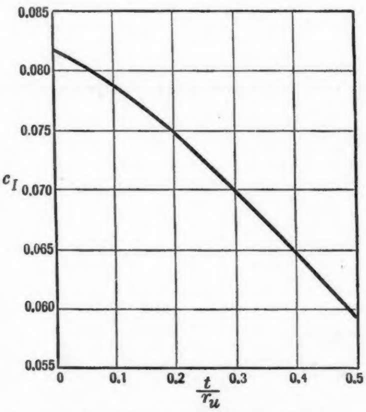

FIG. 3.—MOMENTS OF INERTIA FOR RADIAL SECTIONS.

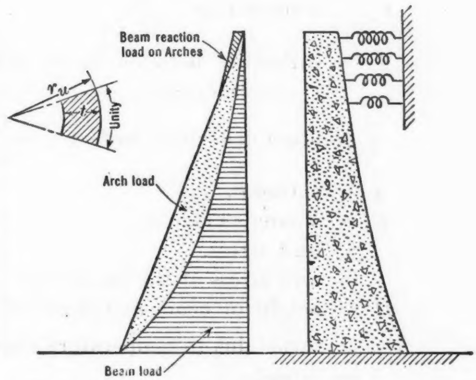

FIG. 4.—DIAGRAMMATIC LOADING FOR VERTICAL BEAM.

(3) The statical conditions of the vertical beams may be considered as very similar to those that probably actually exist. The top arch slices carry very little direct water load, especially if the water does not reach the crest. The result is that they act as a series of yielding supports to the vertical slices, which are thus beams fixed at the bottom and freely supported by a yielding support at the top. Hence, the writer refers to these strips as beams. The conditions of loading may be represented as in Fig. 4, in which helical springs are used to represent the thrust of the top arches. From the deflection curves of Fig. 2, it will be seen that, if the crest deflection is not to be negative, the amount of reaction load possible at the crest is limited, and depends on the shape of the beam. For a parallel-sided beam this reaction cannot exceed 0.2 of the water load without producing negative deflection, while for a beam of triangular section, it cannot exceed 0.075 of the water load. In both cases a triangular distribution of water load is assumed. This is, of course, not quite the case in practice and will alter somewhat the values given.

The writer takes the Salmon Creek Dam as an example to show that the method herein proposed is quite general. The example given merely illustrates the general method and is not a complete analysis. In order to produce exact agreement between arch and cantilever deflections, possibly a half dozen separate trials would be required.

NOTATION

The following notation is used in the paper:

a = top thickness of beam (Fig. 2).

A = a constant $= \frac{x}{y^m}$.

α = coefficient of linear expansion applied to arch material.

b = base thickness of beam.

c = beam deflection coefficient $= \frac{\delta\ E\ b^3}{H^5\ w}$.

C_I = a coefficient $= \frac{I}{t^3}$.

D = difference between mean arch temperature and temperature of construction.

δ = beam deflection, in feet $= \frac{c\ H^5\ w}{E\ b^3}$.

e = extrados.

E = Young's modulus.

f = unit stress.

f' = unit stress at the abutment.

H = height of beam and depth of water.

H_D = thrust due to temperature change, D.

i = intrados.

I = moment of inertia of section.

m = a constant.

$n = \frac{a}{b}$.

p_e = external pressure due to water load = 624 H.

r = mean radius of arch.

r_u = radius of up-stream face.

R = "crest-reaction" load on beam.

t = thickness of arch (Fig. 7).

w = weight of a unit cube of water.

W = total water load on beam.

x = width of canyon at a height, y, above bed-rock.

y = height above bed-rock to the point where x is measured.

DISTRIBUTION OF WATER LOAD

Sections of the proposed dam, all taken radially through the center of curvature of the crest, are set out, and the water-load diagram is drawn against each one. The water load on the central section is now divided into that part which is to be taken by beam action, and that which is to fall on the arches (Fig. 5(c)). This may be done merely by judgment or by Noetzli's preliminary trial method, using the curves of Fig. 2 for beam deflections, and the arch deflection coefficients of Fig. 13 (Appendix I). In using Fig. 2, the non-

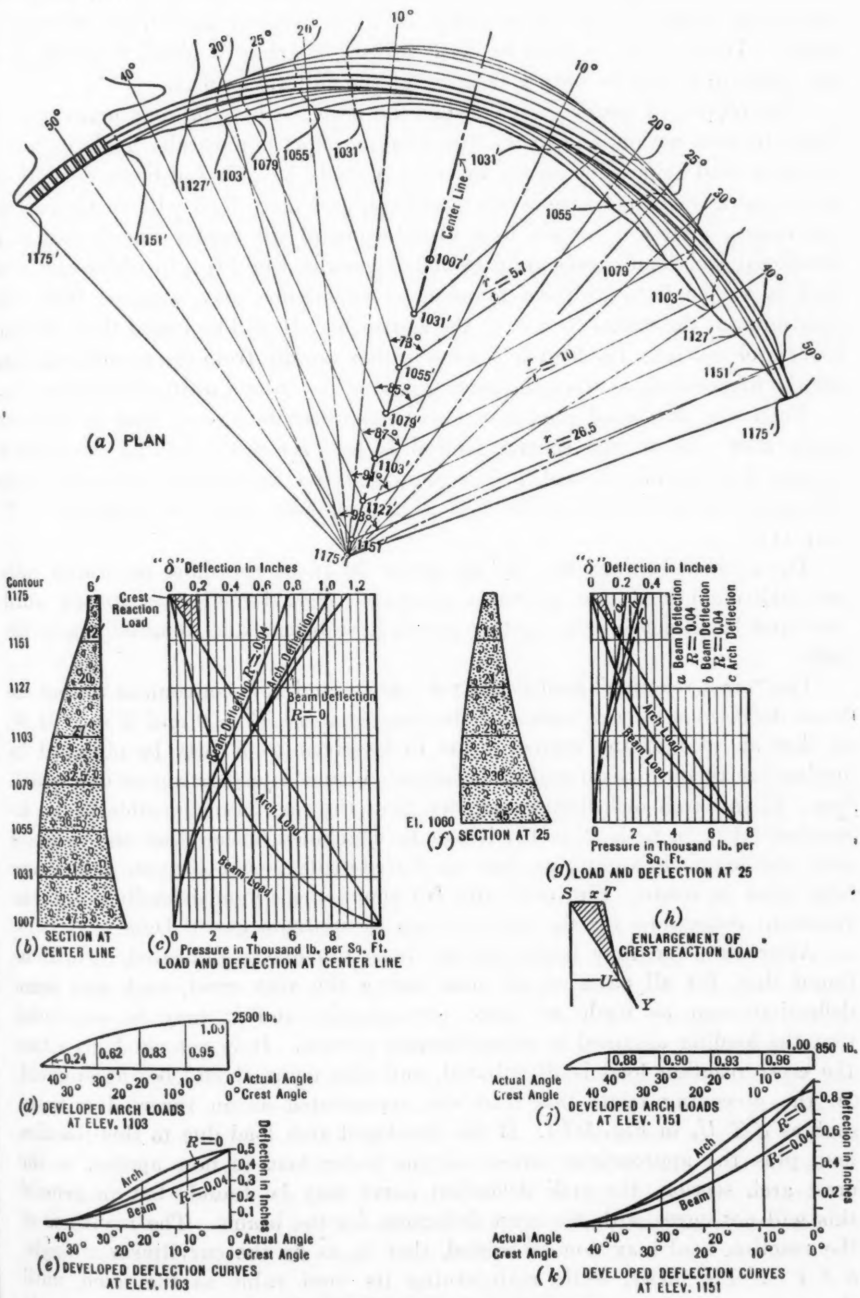

FIG. 5.—DESIGN OF SALMON CREEK DAM.

triangular water load should be replaced by an equivalent triangular load, and the beam section, if not trapezoidal, by an equivalent section of trapezoidal shape. The curves can then be used directly, bearing in mind, however, that the value of w will be only a fraction of the full value, 62.4.

The object, of course, is to design the dam so that the maximum load is taken by arch action, and in the first trial an effort is naturally made to throw as much load as possible on the arches. A study of arch and beam deflections will generally compel a re-division of load, and more load will be thrown on the beams. After a certain load distribution at the center line is assumed, developed arch load curves are set out as shown in Fig. 5(c), in which the arch load is varied in a uniform manner, as experience may suggest, from the maximum at the center to zero at the abutment. It will be found that, for the lower arch sections, the load falls away rather rapidly from the center, whereas for the higher arch sections the loading varies less from a uniform distribution.

From the developed arch load curves, the distribution of load at sections other than that of the central section is now set out. Having assumed a certain distribution of load, it is a simple matter to find the arch and beam deflections at all points of the dam by the methods shown in Appendices II and III.

The arch deflections for the top 20 or 30 ft. of the dam, on which falls the additional load due to "crest reaction", should not be computed until arch and beam deflections at other points have been made to agree reasonably well.

The "crest reaction" load is not yet known, and it is convenient to find the beam deflections for two values of this reaction, say, $R = 0$ and $R = 0.04\ W$, so that an idea of the correct value to be given to R may be obtained by noting the relative beam and arch deflections and interpolating or extrapolating. If the arch deflection is greater than the maximum possible beam deflection (that is, with $R = 0$), it can be concluded, either that the assumed arch loading is excessive, or that its distribution is not correct, and a new trial must be made. Fig. 5(d) and (e) shows the assumed loading, and the resultant deflections for the first trial on the Salmon Creek Dam.

After three or four trials, on the lines previously indicated, it will be found that, for all parts of the dam except the very crest, arch and beam deflections can be made to agree very closely, and it may be concluded that the loading assumed is approximately correct. It is not yet known how the crest reaction load is distributed, and this point should be investigated. In the preceding trials this load was represented as an inverted triangle, such as $S\ T\ U$, in Fig. 5(h). If the developed arch load due to this reaction load plus the appropriate portion of the water load is now applied to the crest arch section, the arch deflection curve may be found; but in general this will not agree with the crest deflection for the beams. The incidence of the reaction load may now be varied, that is, as by the curvilinear triangle, $S\ X\ Y$ in Fig. 5(h), while maintaining its total value as the same, until the crest arch deflections are brought into line with the beam deflections. It must be remembered that the crest arch section subtends the greatest angle

and has the highest value for $\frac{r}{t}$. It is, therefore, extremely sensitive to variations in load distribution, so that it may not be always easy to produce exact agreement of deflections at that point. It will be obvious, however, that such exact agreement is not essential, and the purpose of this last investigation is merely to obtain assurance that the assumed reaction loads are reasonable. This completes the determination of water-load distribution.

Once the exact loading of both beams and arches is known, the determination of the resultant stresses becomes easy. In the case of the beams, the bending moments have been tabulated during the process of finding the deflections, and, therefore, the beam stresses can be written at once. These will be modified by the "no-load stresses", which are due to weight alone, and also, to a small extent, by the arch stresses in a direction at right angles to the beam stresses.

The arch stresses may be found most readily by graphic statics. The values of $\frac{M}{P\,r}$ and $\frac{H}{P}$ are found for the given arch loading as explained in Appendix II, and from these $\frac{M}{H\,r}$ is obtained, thus locating the crown thrust with reference to the center of gravity, G. The load or thrust line can then be drawn, and the value and position of the resultant thrust at any point being known, the stresses can at once be found. These stresses will be slightly modified by the effect of others acting at right angles.

Thus, the stresses caused by water load alone are determined for two extreme assumptions: (1) The arches taking all the load; and (2) the load being shared between beams and arches.

It will generally be found that the beam tensile stresses under Assumption (2) appear to be excessive, which has led some to believe that arch dams must crack, but this is not borne out by practice. It may be stated that if the loads were suddenly applied, such cracks would undoubtedly occur. The application of load in a dam, however, is a slow process, and in some cases it may take several years for the reservoir to be filled completely (Elephant Butte). Hence, the "plastic flow" of concrete is evidently the main saving factor that prevents the tensile stresses from reaching the point indicated by theory. Imperfect fixation between the dam and foundation rock is another, but probably less important factor. Designers realize that stresses must lie between certain known limits, but do not know the exact values. The effect of plastic flow of concrete is a subject requiring investigation, and until such results are available, the writer suggests that actual stresses be assumed by finding the mean between the known limiting stresses. These mean stresses generally will be found to be quite moderate, and might be the basis for determining the actual factor of safety of an arch dam. If undue tension is shown, even with mean stresses, a moderate amount of reinforcement will generally take care of it.

Temperature and Shrinkage

Temperature and shrinkage can be classed together since the effect of shrinkage is identical with that produced by a drop of temperature. For in-

stance, in the latest Italian Government regulations for dams, shrinkage is to be allowed for as a fall of temperature of 18° Fahr. For any given combined temperature and shrinkage effect (Fig. 12, Appendix I) the values of the stresses at the four critical points of the arch can be found if it is assumed that beam action does not restrain the arches. With the same assumption, the arch deflection curve may be traced, either from Fig. 13 which applies to the crown deflection only, or by the use of the deflection influence curves, Fig. 15, by regarding the temperature change (for the purposes of arch deflection only) as equivalent to a uniform water load such as would produce a cylinder stress equal to $E \alpha D$.

This conception of the "equivalent water load" may be misleading as a means of finding arch stresses. In considering deflection and stresses the arch was assumed to deflect to its final position in two steps. First, the ends being assumed free to slide, the whole arch was allowed to shrink. In this position, a certain deflection has occurred, due to temperature, which is completely analogous to that due to water load. This is the "Hauptsystem" of the German writers.* In the case of temperature shrinkage, however, the arch is unstressed, while the arch under water load is subjected to uniform cylinder stress. The second step, that of moving the abutment back to its original position by the application of a horizontal thrust, is identical in regard to its effect on both deflection and stress. The result is that for each step the two cases are analogous as regards deflection, but not as regards stress.

In the particuar case considered, the center line section of Salmon Creek Dam, a temperature change varying uniformly from 10° Fahr. at the base, to 20° Fahr. at the crest, has been assumed. The unrestrained arch deflection curve is shown in Fig. 6(*a*). The "equivalent water load" is also shown. It is obvious that beam action will restrain the arch deflection just as it did in the case of water load, and it might be thought that it would be sufficient to analyze the distribution of this hypothetical water load by the same procedure as was applied in the case of the actual water load which, however, is not correct. In the case of water load, arches and beams deflect until at a certain point each is in equilibrium under the influence of its own share of load and the resisting forces called into play thereby. At a few points in the dam the load on the arch and that on the beam may happen to be equal, but, in general, this is not so. In the case of temperature, however, the deflecting loads on the beams are produced directly by the deflection of the arches, and since action and reaction are equal and opposite, the actual loading on beam and arch is at all points equal.

What actually happens may be analyzed with the help of Fig. 6. Assume that the arch and beam strips are perfectly independent, and allow the temperature variation to occur. The beams will remain unmoved, but the arches will move to a position, such as $L\ M\ N$. The "equivalent water load" producing this deflection is purely imaginary. Both beams and arches are now in a position of rest, which may be taken as a vertical, $L\ Q$, for the beams and $L\ M\ N$ for the arches. Now, allow the interaction between beams and

* "Die Staumauern," by Kalen, 1926.

arches to come into play. These will now come together until they coincide at some position, such as $L\ P\ O$. The beam deflection produced is represented by the difference between the vertical, $L\ Q$, and the line, $L\ P\ O$, while the arch deflection produced is represented by the difference between $L\ M\ N$ and $L\ P\ O$. Both deflections have been produced by equal but opposite loads, and these loads are real, being the system of forces generated by the interaction of arches and beams.

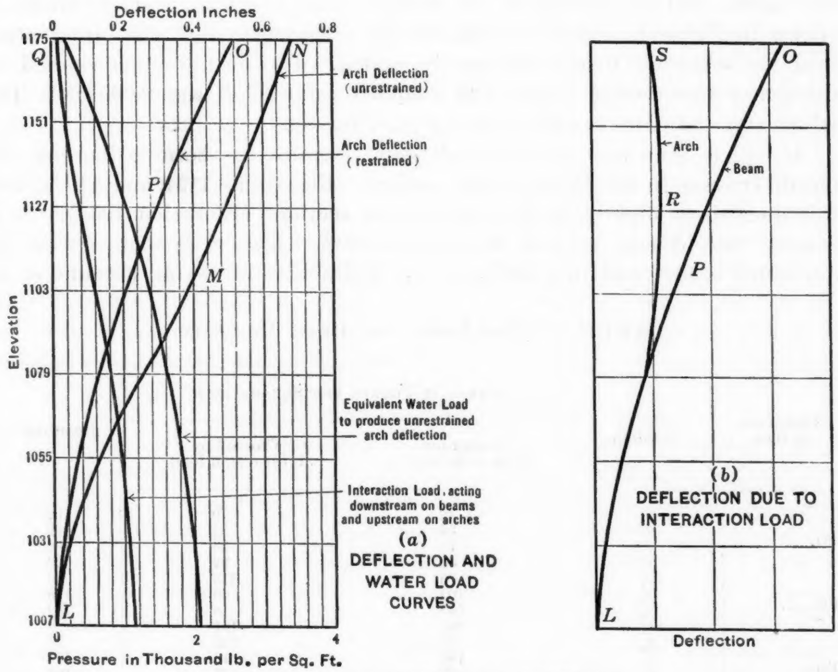

FIG. 6.—TEMPERATURE DEFLECTION CURVES.

Representing arch and beam deflections in the usual way, as in Fig. 6(*b*), it is required to find the real load, which, if applied equally to beams and arches, would cause the former to deflect to the position, $L\ P\ O$, and the latter to the position, $L\ R\ S$. The load which would produce this arch deflection can be determined without trouble, and it then simply remains to check whether or not this same load would produce the required beam deflection. If not, a new trial deflection line, $L\ P'\ O'$, must be taken and the process repeated. After two or three trials the required load can be found.

To determine the stresses is then not difficult. In the case of the beams the only load acting is the real load just found, and the bending moments have already been recorded in the course of the deflection calculation, so that the stresses can be found immediately. In the case of the arches it is necessary to find first the stresses caused by the free deflection (to $L\ M\ N$), and these can be determined at once for the four critical points of the arch from Fig. 12, or for any other point by considering the effect of the thrust, H_D,

which can be readily evaluated. These arch stresses must then be corrected for the effect of the stresses caused by the real "interaction load" which has just been found, and which may be regarded as a water load (although its variation with depth will not, in general, be linear). These second stresses are such as sometimes to ameliorate, and sometimes to aggravate, the effect of the first stresses. In the case considered (drop of temperature), the interaction load shown is to be regarded as a water load acting down stream on the beams, and up stream on the arches. The effect of beam restraint is shown in Table 1, which is based on the assumption that the "interaction load" is uniformly distributed on the arches. The load was determined on the center line section only. The assumed value of E was 2 000 000. The minus sign indicates compression and plus, tension.

It will be seen that the effect of beam restraint has been to increase the tensile stresses in the thinner arch sections (Elevations 1151 and 1103), and to reduce them slightly in the thicker arch sections (Elevation 1055). As a general rule, it may be said that temperature deflection is more serious in restrained beams, and this indicates the desirability of taking account of it.

TABLE 1.—The Effect of Beam Restraint.

Elevation, in feet.	Stress symbol.	Stress, in Pounds per Square Inch.		Resultant.
		Assuming free deflection.	Caused by interaction load.	
1151	f_e	−42	79	37
	f_i	39	53	92
	f_e'	70	36	106
	f_i'	−66	92	26
1103	f_e	−76	115	39
	f_i	109	29	138
	f_e'	190	−8	182
	f_i'	−167	161	−6
1055	f_e	−79	125	46
	f_i	156	−19	137
	f_e'	265	−72	193
	f_i'	−205	193	−12

Combined Effect of Water Load and Uniform Temperature Variation

The simplest way to investigate the combined effect of water load and uniform temperature variation is to consider each one separately and combine the resultant stresses. In this manner it is possible to find, for each part of the dam, the maximum stress occurring under any possible set of conditions, and provision should be made either to sustain this stress, or to reduce it by appropriate alterations in the design.

The most important cases are: (1) Full water load and temperature drop; and (2) no water load and temperature rise. It has been shown that temperature drop, combined with the effect of beam restraint, will cause a fairly general state of arch tension. Hence, under water load, part of the arch compression will be cancelled by this initial tension, and the arch will be more lightly stressed than at the normal temperature. At the same time the

beams will be more heavily loaded. It should be noted that the effect of temperature drop is not merely to throw more of the water load on the beams, since the load diagram now contains extraneous loads not comprised within the water load diagram.

General Discussion

The writer has intentionally confined himself to discussing general principles, since many of the details of their application are already very clearly treated in other excellent contributions to the subject. It will be clear that many details may be modified and refinements may be introduced in the procedure as suggested by the individual judgment of the engineer. For example, no mention has been made of the fact that in the case of a constant-angle dam the beam and arch deflections are in slightly different directions. The correction for this is generally negligible.

It is hoped that the principles outlined will eventually lead to the evolution of a quicker, but more accurate method of arch dam analysis and design. When it is considered that millions of dollars are spent every year in the construction of arch dams, and that if these could be designed so as to utilize fully the strength of the materials specified, probably 20 to 40% of this expenditure might be saved, it will be clear that the subject is one which is worthy of the very closest consideration.

Proposals for Improvements

The widest field for improvements in arch dam design lies, in the writer's opinion, in varying the radius of curvature in order to make the arches better able to sustain the non-uniform load to which they are subjected. As a step in the right direction, may be cited the Italian government regulations which state that it is desirable to vary the curvature as may be required in order to maintain the thrust axial; but they give no indication as to how this should be done. It will be seen from Fig. 17 (Appendix II) that the "load line" departs considerably from the geometrical axis of the arch, and the object of all arch design is to make the two as nearly coincident as possible. This may be done by appropriately varying the curvature of the arch, except that it vitiates at once the relatively simple methods of analysis discussed in this paper. Recourse must then be had to arithmetical and graphical methods of analysis, which in the present case are very laborious. In the case of a dam, the designer is, of course, limited in varying the curvature by the necessity of avoiding overhang. Another proposal investigated by the writer for counteracting the effects of beam action was that of giving the dam a slight overhang in an up-stream direction, at the same time adding a moderate amount of hoop steel to insure stability with reservoir empty. The object was to create sufficient residual compression at the up-stream heel to cancel the beam tension. This is practically equivalent to removing beam action altogether, and thus enabling the dam to act purely as an arch. Such a proposal is not advantageous in the case of a single arch dam but might, in certain cases, be worthy of investigation. The Coolidge Dam,* of the multiple-dome type,

* *Engineering News-Record*, May 27, 1926, p. 865.

appears to take advantage of this residual compression at the up-stream heel, by reason of the beam section being convex up stream.

It has been seen that the excess arch thickness near the crest has a beneficial effect in reducing beam stresses, and the writer considers that additional concrete placed in the crest is a sound investment.

ACKNOWLEDGMENTS

The writer acknowledges his great indebtedness to the work of Professor Guidi, whose method of analysis has been followed very closely in Appendices I and II. He also wishes to thank Mr. R. J. Spowart for the very careful preparation of most of the diagrams and for considerable assistance in the computations, and Mrs. R. A. Sutherland for assistance in preparing the paper for publication.

APPENDIX I

NOTATION

The following notation is used:

α = coefficient of linear expansion of arch material.
B = a coefficient for determining crown deflection under uniform load (see Fig. 13).
c = coefficient for finding true arch stress from cylinder stress.
d = distance from the center line (Fig. 10) of the elastic center of the half arch.
d_x = distance of the antipole, Y, from Axis x-x (with respect to the elastic ellipse of the half arch (see Equation (59)).
D = difference between mean arch temperature and temperature of construction.
δ = crown deflection.
e = extrados.
E = Young's modulus.
f = unit stress.
f' = unit stress at the abutments.
g = elastic weight (in general).
$d\,g$ = elastic weight of the arch element.
g' = elastic weight of the half arch.
G = coefficient of rigidity.
h = rise of arch.
Δh_p = total crown deflection, due to water load.
Δh_D = total crown deflection, due to a uniform variation of temperature, D, from the temperature of construction.
Δh_H = crown deflection, due to the effect of arch thrust.
Δh_k = crown deflection, or change of arch rise, due to uniform arch shortening (neglecting the effect of arch thrust).
H_D = thrust due to temperate change, D.
H_p = horizontal thrust due to water pressure.

i = intrados.

I = moment of inertia of unit arch slice $\left(= \frac{t^3}{12}\right)$.

I_x = moment of inertia of the elastic arch with respect to x-x axis.

I_X = moment of inertia of the elastic arch with respect to X-X axis.

K = a constant depending only on the dimensions of the arch section (see Equation (22)).

l = chord of arch ($A\,B$, Fig. 7) measured from center to center of haunches.

L = length of mean arc.

ΔL = length of arch element.

M = bending moment.

p_e = external pressure due to water load.

ρ = radial semi-axis of the ellipse of elasticity of the arch element.

ρ_1 = tangential semi-axis of the ellipse of elasticity of the arch element (Fig. 8).

r = mean radius of arch.

r_u = radius of up-stream face.

t = thickness of arch (Fig. 7).

T = coefficient for finding stresses due to uniform temperature change.

τ = temperature.

τ_e = difference between extradosal temperature and temperature at which arch was sealed (temperature of construction).

τ_i = difference between intradosal temperature and temperature of construction.

$\Delta \tau$ = difference of temperature between forces.

x = distance from Y-axis of the center of gravity of the arch element.

x-x = axis through the center of gravity of the mean arc and parallel to the chord (see Fig. 7).

X-X = axis parallel to x-x through the center of curvature.

χ = a coefficient relating to the distribution of shear over a cross-section, and depending only on the shape of the section.

y = distance from the x-x axis of the center of gravity of the arch element.

y' = distance from the Axis x-x of the antipole of x-x with respect to elastic ellipse of the arch element.

y_X = distance from Axis X-X of the center of gravity of the arch element.

y_X' = distance from Axis X-X of the antipole of X-X with respect to the elastic ellipse of the arch element.

Z = section modulus.

ϕ = angle measured at the center, O (Fig. 8), from the crown to the arch element.

$\Delta \phi$ = differential angle, ϕ.

ϕ_0 = one-half the central angle measured at the center, O (Fig. 7).

Stress Analysis of an Arch Subjected to Uniform Water Pressure*

A horizontal circular arch strip is taken, with unit height and uniform thickness, and is loaded on the extradosal face by a uniform water load. In such an arch $\left(\text{assuming the ratio, } \frac{\text{radius}}{\text{thickness}}, \text{ to be great}\right)$ the pressure line would correspond with the geometrical arch axis, if the arch were indeformable, and in this case the load on every radial section would reduce to a normal uniform compression of the value, $p_e\,r_u$, or of the value, $p\,r$, in which, r is the mean radius and (see Fig. 7),

$$p = p_e \frac{r_u}{r} \qquad (2)$$

Actually, the arch is shortened by the load to which it is subjected, and the effect of this is evaluated in the following manner. If the abutment exerted simply a thrust, $p\,r$, normal and central, and did not otherwise restrain the section, A, each arch element, ΔL, would be shortened by the amount,

$$d(\Delta L) = \frac{p\,r}{E\,l\,t}\,\Delta L \qquad (3)$$

Hence, the chord, l, would be shortened by the amount,

$$\Delta l = \frac{p\,r}{E\,T}\,l \qquad (4)$$

or, this amount of translation to the right, without rotation, would have to be given to the section, A. Since, however, Section A is effectively immovable, it must develop a supplementary reaction capable of annulling the displacement mentioned, which, according to the elastic theory, has for its line of action the axis, x-x, passing through the elastic center of gravity, G, of the arch, and parallel to the chord, $A\,B$. This reaction is directed outward (that is, negative) and its value is given by,

$$H_p = \frac{\frac{p\,r}{E\,t}\,l}{I_x} \qquad (5)$$

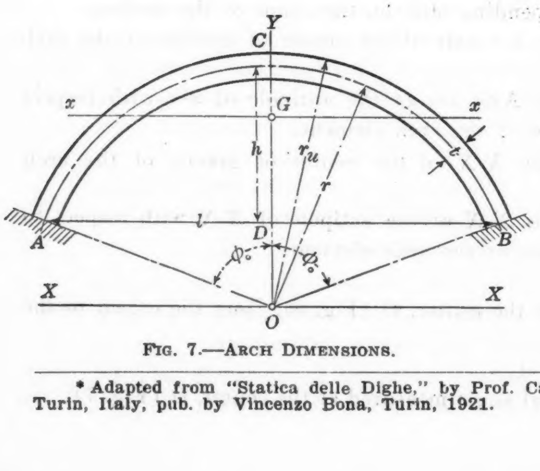

Fig. 7.—Arch Dimensions.

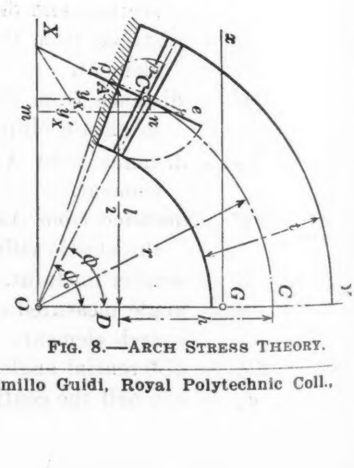

Fig. 8.—Arch Stress Theory.

* Adapted from "Statica delle Dighe," by Prof. Camillo Guidi, Royal Polytechnic Coll., Turin, Italy, pub. by Vincenzo Bona, Turin, 1921.

Now, the elastic weight of the arch element is given by,

$$dg = \frac{dL}{E\,I} \qquad (6)$$

Hence, dg is proportional to dL, and, therefore, the elastic center of gravity, G, of the arch corresponds with the center of gravity of its geometrical axis. Referring to Fig. 7:

$$I_X = \frac{12}{E\,t^3} \int y\,y'\,d\,L \qquad (7)$$

This is the moment of inertia with respect to the axis, X, passing through the center of curvature. Then, transferring to Axis x:

$$I_x = \frac{12}{E\,t^3} \left(\int y_X\,y_X'\,d\,L - (\text{line } O\,G)^2 \int d\,L \right) \qquad (8)$$

Fig. 8 represents the half arch, with an exaggerated ratio of radius to thickness for the sake of clearness.

For the arch element, ΔL (Fig. 8):

$$\rho^2 = \frac{1}{12}\,t^2 \qquad (9)$$

and,

$$\rho_1^{\,2} = \frac{(\Delta\,L)^2}{12} + \chi\,\frac{E}{G}\,\rho^2 \qquad (10)$$

Taking $\frac{E}{G} = \frac{5}{2}$, and remembering that, for a rectangular section, $\chi = \frac{6}{5}$:

$$\rho_1^{\,2} = \frac{(\Delta\,L)^2}{12} + 3\,\rho^2 \qquad (11)$$

When $\Delta\,L$ becomes infinitesimal,

$$\rho_1^{\,2} = 3\,\rho^2 = \frac{t^2}{4}$$

For any arch element (Fig. 8):

$$y_X = r \cos \phi \qquad (12)$$

$$y_X' = (\text{line } m\,n) + (\text{line } n\,e) = (\text{line } O\,C \cos \phi) + (\text{line } C\,e \sin \phi)$$

$$= \left(r + \frac{\rho^2}{r}\right) \cos \phi + \frac{\rho_1^{\,2}}{r \cot \phi} \sin \phi = \left(r + \frac{t^2}{12\,r}\right) \cos \phi + \frac{t^2}{4\,r}\,\frac{\sin^2 \phi}{\cos \phi} \qquad (13)$$

Also,

$$d\,L = r\,d\,\phi \qquad (14)$$

and,

$$(\text{line } O\,G) = \frac{r \sin \phi_0}{\phi_0} \qquad (15)$$

Hence,

$$\int y_X\,y_X'\,d\,L = 2\,r^3 \left[\left(1 + \frac{t^2}{12\,r^2}\right) \int_0^{\phi_0} \cos^2 \phi\,d\,\phi + \frac{t^2}{4\,r^2} \int_0^{\phi_0} \sin^2 \phi\,d\,\phi \right]$$

$$= r^3 \left[\left(1 + \frac{t^2}{12\,r^2}\right) \left(\phi_0 + \frac{\sin 2\,\phi_0}{2}\right) + \frac{t^2}{4\,r^2} \left(\phi_0 - \frac{\sin 2\,\phi_0}{2}\right) \right] \qquad (16)$$

Also,

$$(\text{line } (O\ G)^2) \int d\ L = 2\ r^3 \frac{\sin^2 \phi_0}{\phi_0} \quad \ldots\ldots\ldots\ldots (17)$$

Hence,

$$I_x = \frac{12\ r^3}{E\ t^3} \left[\left(1 + \frac{t^2}{12\ r^2}\right) \left(\phi_0 + \frac{\sin 2\ \phi_0}{2}\right) + \frac{t^2}{4\ r^2} \left(\phi_0 - \frac{\sin 2\ \phi_0}{2}\right) - \frac{2 \sin^2 \phi_0}{\phi_0} \right] \quad \ldots\ldots\ldots\ldots (18)$$

From geometry (Fig. 8):

$$\phi_0 = \frac{L}{2\ r} \quad \ldots\ldots\ldots\ldots (19)$$

$$\frac{\sin 2\ \phi_0}{2} = \sin \phi_0 \cos \phi_0 = l\ \frac{r - h}{2\ r^2} \quad \ldots\ldots\ldots\ldots (20)$$

$$\frac{2 \sin^2 \phi_0}{\phi_0} = \frac{l^2}{r\ L} \quad \ldots\ldots\ldots\ldots (21)$$

Hence,

$$I_x = \frac{6\ r^2\ l}{E\ t^3} \left[\frac{L}{l} + \frac{r - h}{r} - \frac{2\ l}{L} + \frac{t^2}{6\ r^2} \left(\frac{2\ L}{l} - \frac{r - h}{r} \right) \right] \quad \ldots\ldots (22)$$

Calling the expression in square brackets, K_0, this reduces to,

$$I_x = \frac{6\ K_0\ r^2\ l}{E\ t^3} \quad \ldots\ldots\ldots\ldots (23)$$

Equation (5) now reduces to,

$$H_p = p\ r\ \frac{t^2}{6\ K_0\ r^2} \quad \ldots\ldots\ldots\ldots (24)$$

On account of this outward supplementary reaction, the line of pressure is displaced outward above the axis, x, and inward below this axis. The algebraic sum of the areas between the new pressure line and the geometric axis of the arch is almost exactly zero.

Maximum Stresses Due to Water Load

The sections of the arch most heavily stressed are those at the crown and at the abutment. Assuming the trapezium law to be correct (Fig. 8):

$$f_e = -\frac{p\ r}{t} - \frac{6\ H_p \left[(\text{line } G\ C) - \frac{t}{6} \right]}{t^2} \quad \ldots\ldots\ldots\ldots (25)$$

$$f_i = -\frac{p\ r}{t} + \frac{6\ H_p \left[(\text{line } G\ C) + \frac{t}{6} \right]}{t^2} \quad \ldots\ldots\ldots\ldots (26)$$

$$[\text{line } G\ C] = r \left(1 - \frac{\sin \phi_0}{\phi_0}\right) = r\ \frac{L - l}{L} \quad \ldots\ldots\ldots\ldots (27)$$

Substituting the value of H_p from Equation (24):

$$f_e = -p\left[\frac{r}{t} + \frac{1}{K_0}\left(\frac{L-l}{L} - \frac{t}{6\,r}\right)\right] \quad \text{(28)}$$

$$f_i = -p\left[\frac{r}{t} - \frac{1}{K_0}\left(\frac{L-l}{L} + \frac{t}{6\,r}\right)\right] \quad \text{(29)}$$

also,

$$f_e' = -\frac{p\,r}{t} + \frac{6\,H_p}{t^2}\left[(\text{line } G\,D) + \frac{t}{6}\cos\phi_0\right] \quad \text{(30)}$$

$$f_i' = -\frac{p\,r}{t} - \frac{6\,H_p}{t^2}\left[(\text{line } G\,D) - \frac{t}{6}\cos\phi_0\right] \quad \text{(31)}$$

Since (Fig. 8),

$$(\text{line } G\,D) = (\text{line } G\,O) - (\text{line } D\,O) = \frac{r\sin\phi_0}{\phi_0} - r\cos\phi_0$$

$$= r\left(\frac{l}{L} - \frac{r-h}{r}\right) \quad \text{(32)}$$

therefore,

$$f_e' = -p\left\{\frac{r}{t} - \frac{1}{K_0}\left[\frac{l}{L} - \frac{r-h}{r}\left(1 - \frac{t}{6\,r}\right)\right]\right\} \quad \text{(33)}$$

$$f_i' = -p\left\{\frac{r}{t} + \frac{1}{K_0}\left[\frac{l}{L} - \frac{r-h}{r}\left(1 + \frac{t}{6\,r}\right)\right]\right\} \quad \text{(34)}$$

The value determined from Equation (34) is always the greatest of the four stresses evaluated. Hence, Noetzli's assumption of a two-hinged arch errs not only qualitatively, but quantitatively, in that it assumes the crown stresses to be the deciding factor.

Stresses Due to Temperature Variation

In general, the up-stream and down-stream faces of a dam will not be at the same temperature, nor will they be at the temperature of construction. Hence, stresses are introduced due both to the mean variation from the temperature of construction and to the difference of temperature between the two faces. Calling these τ_e and τ_i for the extrados and intrados, respectively, and assuming a linear variation between the two, it is safe to assume that the values, τ_e and τ_i, have been reached by: (1) A uniform variation of temperature equal to $\frac{\tau_e + \tau_i}{2} = D$ from the temperature of construction; and (2) a local variation of $\frac{\tau_e - \tau_i}{2}$ at the extrados, and $-\frac{\tau_e + \tau_i}{2}$ at the intrados.

Assuming a linear variation between τ_e and τ_i may be quite legitimate for thin arches, such as small dams, or multiple-arch dams, but will not in general be even approximate for large arch dams.

For example, supposing both τ_e and τ_i represent rises in temperature, Fig. 9 shows (to an exaggerated scale) the effect on a small arch element. Assuming first that one abutment of the arch is free, the terminal face of the

arch element, EF, would assume the position, $E' F'$, where $E E' = \alpha \, \tau_e \, \Delta$ L and $F F' = \alpha \, \tau_i \, \Delta \, L$.

This deformation consists of two parts: (1) A translation, $d \, (\Delta \, L)$, due to the general rise of temperature, $\frac{\tau_e + \tau_i}{2}$; and (2) a rotation expressed by:

$$d \, (\Delta \, \phi) = \frac{\alpha \, (\tau_e - \tau_i)}{2} \, \Delta \, L = \frac{\alpha \, \Delta \, \tau \, \Delta \, L}{t} \, \ldots\ldots\ldots\ldots (35)$$

The first deformation taking place over the whole arch, would produce a translation of the free abutment of the dam by a distance,

$$\alpha \, \frac{\tau_e + \tau_i}{2} \, l = \alpha \, D \, l$$

Since the abutment is effectively immovable, the thrust exerted along the axis, x, in an inward direction, has the value,

$$H_D = \frac{\alpha \, D \, l}{I_x} \, \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

and substituting the value of I_x, Equation (23),

$$H_D = \frac{t^3}{6 \, K_0 \, r^2} \, E \, \alpha \, D \, \ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)$$

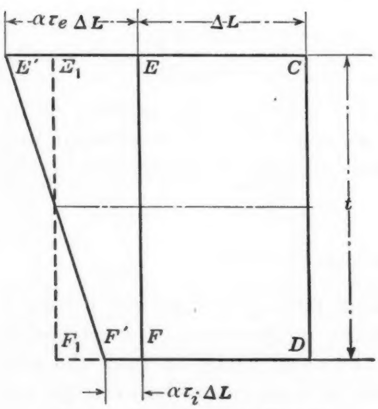

FIG. 9.—TEMPERATURE THEORY.

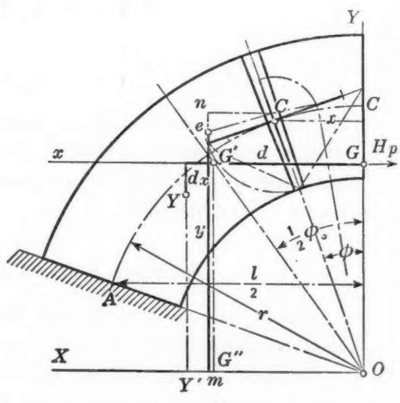

FIG. 10.—ARCH DEFLECTION THEORY.

The second deformation, due to the local temperature variations of the two faces, would cause the end of the arch, if free—and hence the elastic center of gravity, G, which is always considered to be rigidly connected thereto—to undergo a rotation,

$$\Delta \, \phi = \alpha \, \Delta \, \tau \sum \frac{\Delta \, L}{t} \, \ldots\ldots\ldots\ldots\ldots\ldots (38)$$

Hence, the point, G, would be translated a distance, the co-ordinates of which are,

$$\Delta \, x = \alpha \, \Delta \, \tau \sum \frac{\Delta \, L}{t} \, y \, \ldots\ldots\ldots\ldots\ldots\ldots (39)$$

$$\Delta \, y = \alpha \, \Delta \, \tau \sum \frac{\Delta \, L}{t} \, x \, \ldots\ldots\ldots\ldots\ldots\ldots (40)$$

The term, t, being constant, may be taken outside the summation sign, and since G coincides with the center of gravity of the arch axis,

$$\sum y \, \Delta L = \sum x \, \Delta L = 0 \qquad (41)$$

hence,

$$\Delta x = \Delta y = 0 \qquad (42)$$

Therefore, the point, G, does not move, which means that if the end of the arch were free, it would rotate about G. Since the end of the arch is fixed, it must develop a moment, given by Equation (43),

$$M = E \frac{\Sigma \, d(\Delta \phi)}{\sum \frac{\Delta L}{I}} = \frac{E \, I \, \alpha \, \Delta \tau}{t} \qquad (43)$$

and, since,

$$I = \frac{t^3}{12}$$

$$M = \frac{E \, \alpha \, \Delta \tau}{12} \, t^3 \qquad (44)$$

To find the stresses caused by the general and local variations of temperature is now easy. The former will bear, numerically, the same ratio to the stresses due to water load as H_D bears to H_p; that is, as $E \, \alpha \, D \, \tau$ is to $p \, r$. Algebraically, they will be subtracted from or added to those, according as the general temperature variation is a rise or fall. The stresses due to local temperature variation are equal for all arch sections and are given by,

$$\pm \frac{M}{Z} = \frac{\frac{E \, \alpha \, \Delta \tau}{12} \, t^2}{\frac{t^2}{6}} = \frac{E \, \alpha \, \Delta \tau}{2} \, t \qquad (45)$$

The negative sign is given to the stress that occurs at the face where the temperature is the greater, and the positive sign to the stress at the other face.

It is convenient to combine Equations (33) and (34) for stress into one expression giving the stress at the four critical points (see Equation (23)).

For brevity, let,

$$K_e = \frac{1}{K_0} \left[\frac{L - l}{L} - \frac{t}{6 \, r} \right] \qquad (46)$$

$$K_i = \frac{1}{K_0} \left[\frac{L - l}{L} + \frac{t}{6 \, r} \right] \qquad (47)$$

$$K_e' = \frac{1}{K_0} \left[\frac{l}{L} - \frac{r - h}{r} \left(1 - \frac{t}{6 \, r} \right) \right] \qquad (48)$$

$$K_i' = \frac{1}{K_0} \left[\frac{l}{L} - \frac{r - h}{r} \left(1 + \frac{t}{6 \, r} \right) \right] \qquad (49)$$

Then, the crown stresses are:

$$f_e = -p\left(\frac{r}{t} + K_e\right) + E\,\alpha\left(D\,\frac{t}{r}\,K_e - \frac{\Delta\,\tau}{2}\right) \ldots\ldots\ldots (50)$$

$$f_i = -p\left(\frac{r}{t} - K_i\right) - E\,\alpha\left(D\,\frac{t}{r}\,K_i - \frac{\Delta\,\tau}{2}\right) \ldots\ldots\ldots (51)$$

and the abutment stresses are:

$$f_e' = -p\left(\frac{r}{t} - K_e'\right) - E\,\alpha\left(D\,\frac{t}{r}\,K_e' + \frac{\Delta\,\tau}{2}\right) \ldots\ldots\ldots (52)$$

$$f_i' = -p\left(\frac{r}{t} + K_i'\right) + E\,\alpha\left(D\,\frac{t}{r}\,K_i' + \frac{\Delta\,\tau}{2}\right) \ldots\ldots\ldots (53)$$

The term, $\frac{\Delta\,\tau}{2}$, should be used as it stands only for relatively thin arches. Table 2 is useful in showing at a glance the direction of the stress due to any given condition.

TABLE 2.—Direction of Stress for Any Given Condition (Compression —, Tension +).

Section.	Water Load Stresses.		Variation of Temperature.			
	$p\,r$.	H_p.	D.		$\Delta\,\tau$.	
			Rise.	Fall.	Rise.	Fall.
Crown:						
Extrados	—	—	+	—	—	+
Intrados	—	+	—	+	+	—
Abutment:						
Extrados	—	+	—	+	—	+
Intrados	—	—	+	—	+	—

Crown Deflection Due to Water Load

Crown deflection takes place in the direction of the axis, Y, and may be regarded as the sum of that due to the uniform shortening of the arch under the stress, $\frac{p\,r}{t}$, and that due to the reaction, H_p, given by Equation (24).

The first deflection is obviously equal to,

$$\Delta\,h_k = \frac{p\,r}{E\,t}\,h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (54)$$

and the second, $\Delta\,h_H$, must now be determined.

Consider the half arch fixed at one abutment and loaded on the crown section by the reaction, H_p, passing through the elastic center of gravity, G, of the whole arch (see Fig. 10). According to the theory of the ellipse of elasticity, the deflection, $\Delta\,h_H$, is given by the expression,

$$\Delta\,h_H = H_p\,g'\,d\,d_x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (55)$$

in which,

$$g' = \frac{1}{2}\,\frac{L}{E\,I} = \frac{6\,L}{E\,t^3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (56)$$

The elastic center of gravity, G', of the half arch evidently falls on the x-x axis passing through G, and,

$$(\text{line } O\,G') = r\,\frac{r \sin \frac{\phi_0}{2}}{r\,\frac{\phi_0}{2}} = \frac{r \sin \frac{\phi_0}{2}}{\frac{\phi_0}{2}} \quad \text{.............. (57)}$$

Whence,

$$d = (\text{line } G\,G') = (\text{line } O\,G') \sin \frac{\phi_0}{2} = \frac{r \sin^2 \frac{\phi_0}{2}}{\frac{\phi_0}{2}} \quad \text{......... (58)}$$

Also,

$$d_x = (\text{line } G'\,G'') - (\text{line } Y\,Y') \quad \text{................ (59)}$$

Now,

$$(\text{line } G'\,G'') = (\text{line } O\,G) = r\,\frac{\sin \phi_0}{\phi_0} \quad \text{.............. (60)}$$

and,

$$(\text{line } Y\,Y') = \frac{\int_0^{\phi_0} x\,y\,d\,L}{r\,\phi_0\,r\,\frac{\sin^2 \frac{\phi_0}{2}}{\frac{\phi_0}{2}}} \quad \text{.................. (61)}$$

The expression to be integrated may be reduced as follows:

$$x\,y\,d\,L = r\,d\,\phi \sin \phi\,[(\text{line } m\,n) - (\text{line } e\,n)]$$
$$= r^3 \left(1 - \frac{t^2}{6\,r^2}\right) \sin \phi \cos \phi\,d\,\phi \quad \text{................ (62)}$$

Hence, from Equations (62) and 61):

$$Y\,Y' = r\left(1 - \frac{t^2}{6\,r^2}\right) \frac{\int_0^{\phi_0} \sin \phi \cos \phi\,d\,\phi}{2 \sin^2 \frac{\phi_0}{2}}$$
$$= \frac{r}{2}\left(1 - \frac{t^2}{6\,r^2}\right)(1 + \cos \phi_0) \quad \text{................. (63)}$$

and,

$$d_x = r\,\frac{\sin \phi_0}{\phi_0} - \frac{r}{2}\left(1 - \frac{t^2}{6\,r^2}\right)(1 + \cos \phi_0) \quad \text{........... (64)}$$

From geometry (Fig. 10), Equations (58) and (64) may be simplified to:

$$d = \frac{2\,h\,r}{L} \quad \text{.......................... (65)}$$

and,

$$d_x = l\left[\frac{r}{L} - \frac{l}{8\,h}\left(1 - \frac{t^2}{6\,r^2}\right)\right] \quad \text{................. (66)}$$

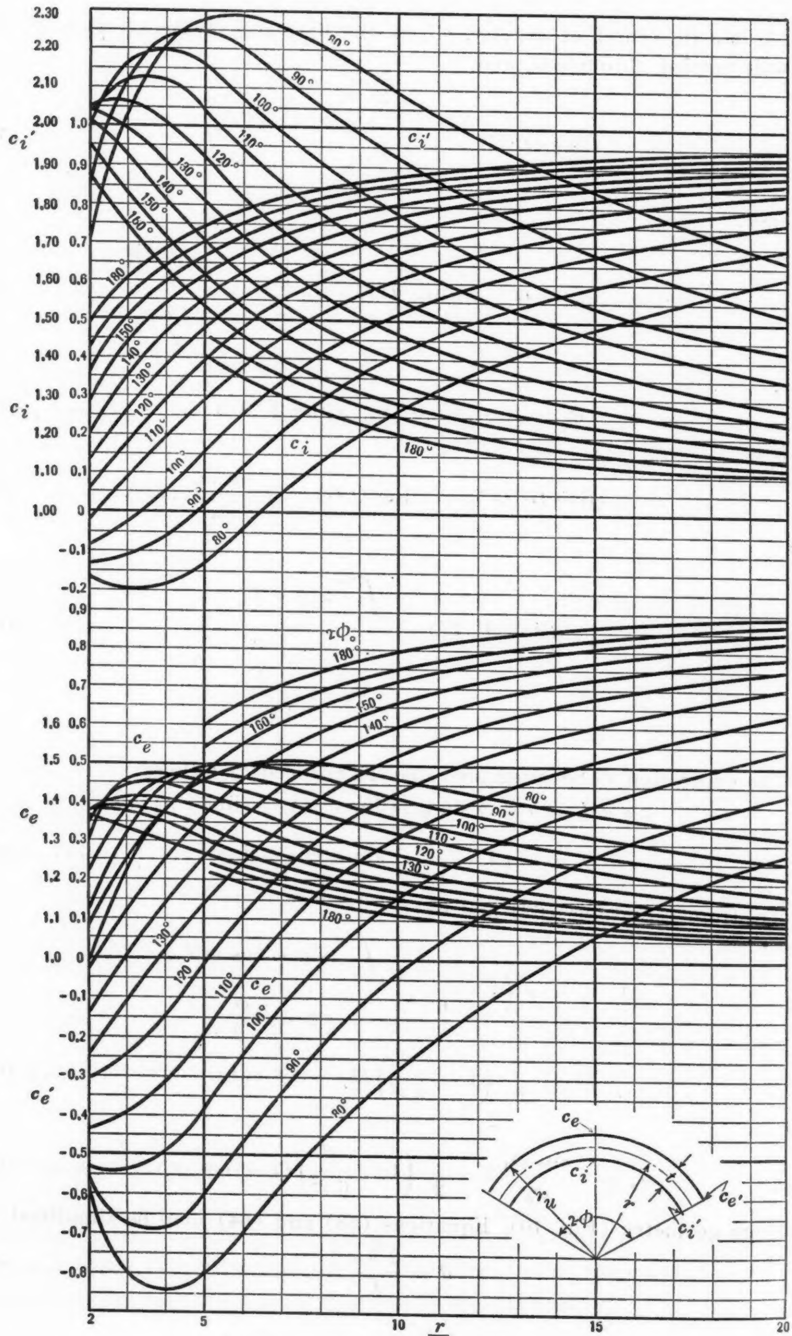

FIG. 11.—WATER LOAD STRESS COEFFICIENTS.

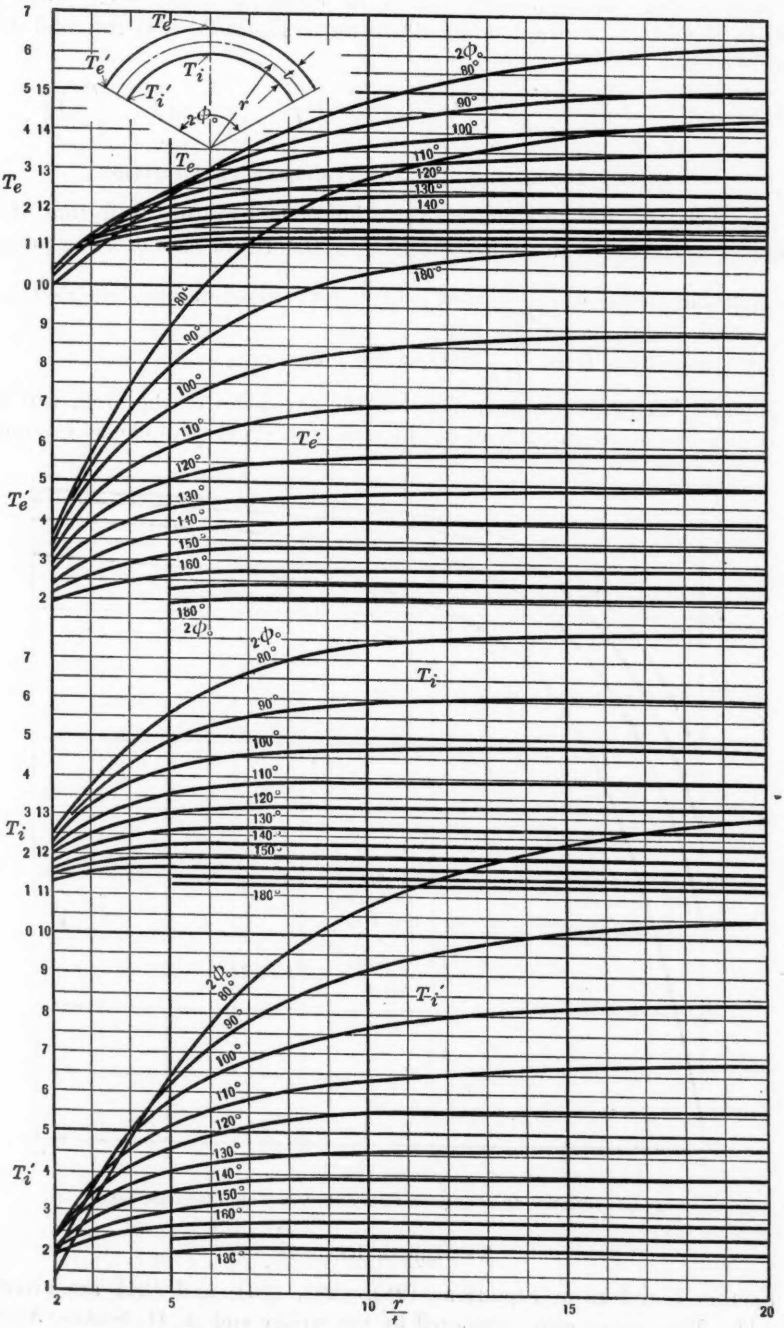

FIG. 12.—TEMPERATURE STRESS COEFFICIENTS.

Substituting the expressions given in Equations (24), (52), (65), and (66) in Equation (55) and adding the latter to Equation (54) the total deflection becomes:

$$\Delta h_p = \Delta h_k + \Delta h_H = \frac{p}{E}\frac{r}{t} h \left\{ 1 + \frac{1}{K_0}\left[\frac{2\,l}{L} - \frac{l^2}{4\,h\,r}\left(1 - \frac{t^2}{6\,r^2}\right)\right]\right\} \cdot (67)$$

Crown Deflection Due to Temperature Variation

The deflection due to a general variation of temperature from that of construction, is caused by the unit shortening, $\alpha\, D$. In the case of water pressure, the unit shortening was $\frac{p\,r}{E\,t}$; hence, to find the temperature deflection, $\Delta\, h_D$, simply substitute $\alpha\, D$ for $\frac{p\,r}{E\,t}$ in Equation (67).

A variation of temperature between the two faces of the arch, will obviously cause no resultant deflection, since the two effects will cancel each other.

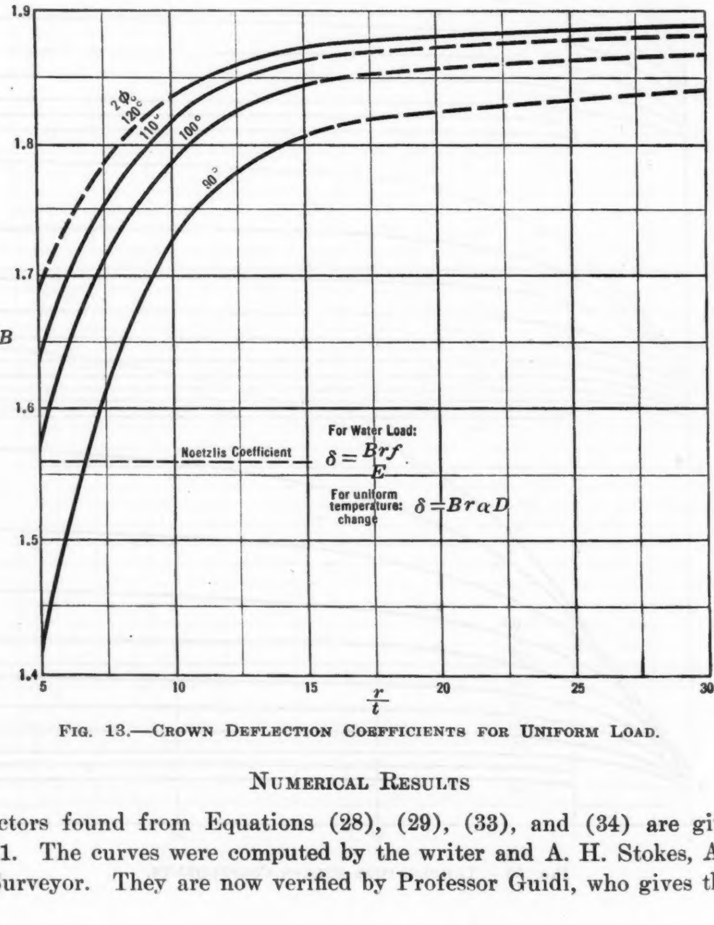

Fig. 13.—Crown Deflection Coefficients for Uniform Load.

Numerical Results

Factors found from Equations (28), (29), (33), and (34) are given in Fig. 11. The curves were computed by the writer and A. H. Stokes, Authorized Surveyor. They are now verified by Professor Guidi, who gives them in

a less convenient form, although covering a wider range of angles.* By means of these curves the correct arch stress due to uniform water load can be found, for the four critical points indicated, in terms of the cylinder stress. To find the correct maximum arch stress (neglecting vertical beam action) multiply the stress, as found by Equation (1), by the appropriate coefficient.

Similarly, the factors, T, given in Fig. 12, make it readily possible to determine the critical stresses due to a uniform change of temperature. The stress due to a uniform change of temperature, D, from the temperature of construction, is given by,

$$f_T = T\,E\,\alpha\,D\,\frac{t}{r} \qquad (68)$$

The crown deflection due either to uniform water pressure or uniform temperature change can readily be found from the coefficients given in Fig. 13. These vary from about $B = 1.4$ to $B = 1.9$ for the angles shown and take the place of the uniform factor, 1.56, given by Noetzli in his Equations (13*a*) and (13*b*).† The reason that Noetzli's factor is generally too low, is that shear was neglected, the arch being assumed to be free-ended.

APPENDIX II

Notation

The notation used is as follows:

β = angle measured at the center (O, Fig. 14) from the crown to the point of application of P.

δ = arch deflection at the point under consideration.

E = Young's modulus.

η = distance of the point of intersection of P and the arch axis from a horizontal through C (Fig. 14(a)).

h_1 = the rise of an arch subtending an angle, $2\,(\phi_0 - \beta)$.

H = horizontal thrust.

I = moment of inertia.

I_x = moment of inertia of the elastic semi-arch with respect to Axis x-x.

K = a coefficient.

K_0 = a constant depending only on the dimensions of the arch section.

l = chord of arch ($A\,B$, Fig. 15) measured from center to center of haunches.

L = length of mean arc.

L_1 = length of arc comprised between two symmetrical forces, P.

M = bending moment added to counteract the translation of the arch thrust, H, from the crown to the axis through the elastic center, G', of the half arch.

* "Statica delle Dighe per Laghi Artificiali," Second Edition, 1926.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 12.

N = a coefficient $= \dfrac{\delta\, E\, I}{P_r^{\,3}}$ (see Fig. 15).

p = pressure due to water load.

p_e = external pressure due to water load.

P = one of a pair of equal radial forces applied to the arch at equal angles, β, from the center line.

ρ = radial semi-axis of the ellipse of elasticity of the arch element.

ρ_1 = tangential semi-axis of the ellipse of elasticity of the arch element (Fig. 8).

q = distance from the line of action of the thrust, H, of the antipole, with respect to the elastic ellipse of the arch element, of the radius through the point under consideration (Line ND, Fig. 14(d)).

r = mean radius of arch.

r_u = radius of up-stream face.

s = Line SD, Fig. 14(d) = distance of antipole defined by q from the line of action of the force, P.

t = thickness of arch.

x-x = axis through the center of gravity of the mean arc and parallel to the chord (see Fig. 14).

X-X = axis parallel to x-x through the center of curvature.

y = distance from the axis, x-x, of the antipole of action of P with respect to the ellipse of elasticity of the arch element.

y_X = distance from the axis, X-X, of the antipole of action of P, with respect to the ellipse of elasticity of the arch element referred to the X-X-axis.

z = distance of the center of the arch element from the line of action of the force, P.

z_1 = distance of the center of the arch element from the radius through the point under consideration.

ζ = distance from the center line, of the point of intersection of P and the mean arc, or arch axis.

ϕ = angle measured at the center, O (Fig. 14), from the crown to the arch element.

ϕ_o = one-half the central angle measured at the center, O (Fig. 14).

ψ = angular distance from the center line of the point the deflection of which is to be found.

Deflection Produced by Non-Uniform Water Pressure

The hypothesis generally accepted in the design of arch dams is that the water pressure is uniform over any given horizontal arch slice, but this does not correspond with fact. This conclusion, if it holds at all, can do so only for a dam in a site of rectangular section. In the vast majority of cases, with a **V**- and **U**-shaped site, the reverse is the case; that is, the pressure on the arch slice is greatest in the center. This is amply borne out by the recent measure-

ments on the Stevenson Experimental Dam.* The arch dam acts as a plate, and the pressure on any given arch slice increases from the crown to the abutments, which is the only place that it has full value due to the head of water.

It is of interest to examine the effect of such unequal load distribution on the stresses and deflections produced. Consider an arch slice of constant section and unit height, as shown in Fig. 14, and determine first the thrust deflection at the crown produced by two equal and symmetrical radial forces, P. The crown section will obviously not rotate nor be displaced in a direction normal to the radius through the crown section. Hence, take the half arch, Fig. 14(a), solidly fixed at A and loaded by the force, P, and by the thrust, H, produced by the other half arch similarly loaded. By reason of symmetry the thrust, H, is normal to the radius, $O\,C$, and may be translated to pass through the elastic center, G', of the half arch by adding the moment, M, at G'.

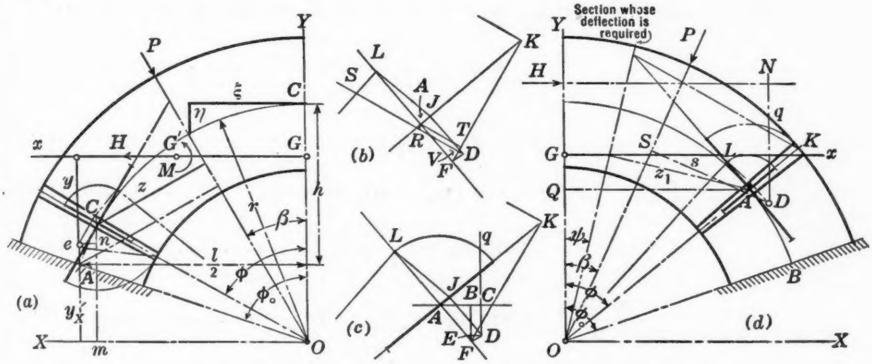

FIG. 14.—ARCH THEORY FOR CONCENTRATED LOADS.

It is then possible to write two equations expressing the fact that the rotation of the crown section is nil, as is also the deflection (in a direction normal to the Y-axis) of the elastic center of gravity, G', which is considered rigidly connected with the crown, C.

The first equation is,

$$-\frac{L}{2}\,M + P\int_{\beta}^{\phi_0} z\,d\,L = 0 \qquad (68)$$

Noting that $d\,L = r\,d\,\phi$ and integrating, this equation may be written:

$$M = 2\,\frac{P\,r^2}{L}\,[1 - \cos\,(\phi_0 - \beta)] = 2\,P\,r\,\frac{h_1}{L} \qquad (69)$$

The other equation is,

$$-H\,I_x + P\int_{\beta}^{\phi_0} z\,y\,d\,L = 0 \qquad (70)$$

Representing the elementary elastic weight simply by $d\,L$ (to which it is proportional), the value of I_x is given by,

$$I_x = \frac{l\,r^2\,K_0}{4} \qquad (71)$$

* As reported in *Engineering News-Record*, November 18, 1926, p. 828.

Furthermore,

$$z = r \sin(\phi - \beta) \quad \ldots\ldots(72)$$

and,

$$y = (\text{line } O\ G) - [(\text{line } m\ c) - (\text{line } n\ c)] = \frac{r \sin \phi_0}{\phi_0}$$

$$- \left[\left(r + \frac{\rho^2}{r}\right) \cos \phi \ \frac{{\rho_1}^2}{r \tan(\phi - \beta)} \sin \phi\right] \quad \ldots\ldots(73)$$

Hence,

$$\int_\beta^{\phi_0} z\ y\ d L = r^3 \left[\frac{\sin \phi_0}{\phi_0} \int_\beta^{\phi_0} \sin(\phi - \beta)\ d\ \phi\right.$$

$$- \left(1 + \frac{\rho^2}{r^2}\right) \int_\beta^{\phi_0} \sin(\phi - \beta) \cos \phi\ d\ \phi - \frac{{\rho_1}^2}{r^2} \int_\beta^{\phi_0} \sin \phi \cos(\phi - \beta)\ d\ \phi. \quad (74)$$

To simplify the integration, the following relationships are convenient:

$$\rho^2 = \frac{t^2}{12} \quad \ldots\ldots(75)$$

$${\rho_1}^2 = \frac{t^2}{4} \quad \ldots\ldots(76)$$

$$r\,[1 - \cos(\phi_0 - \beta)] = h_1 \quad \ldots\ldots(77)$$

$$\sin^2 \phi_0 = \frac{l^2}{4\ r^2} \quad \ldots\ldots(78)$$

$$\frac{\sin 2\ \phi_0}{2} = \sin \phi_0 \cos \phi_0 = l\ \frac{r - h}{2\ r^2} \quad \ldots\ldots(79)$$

$$\sin \beta = \frac{\zeta}{r} \quad \ldots\ldots(80)$$

$$\phi_0 - \beta = \frac{L - L_1}{2\ r} \quad \ldots\ldots(81)$$

and,

$$\cos \beta = \frac{r - \eta}{r} \quad \ldots\ldots(82)$$

Substituting Equations (75) to (82) in Equation (74):

$$\int_\beta^{\phi_0} z\ y\ d\ L = \frac{r^3}{2} \left[2\ \frac{l}{L}\ \frac{h_1}{r} - \left(1 - \frac{t^2}{6\ r^2}\right) \left(\frac{r - \eta}{r}\ \frac{l^2}{4\ r^2} - \frac{\zeta}{r}\ \frac{l}{2\ r}\ \frac{r - h}{r}\right)\right.$$

$$\left. + \left(1 + \frac{t^2}{3\ r^2}\right) \frac{\zeta}{r}\ \frac{(L - L_1)}{2\ r}\right]$$

Hence,

$$H = \frac{2\ P\ \dfrac{2\ h_1}{L} + \left(1 + \dfrac{t^2}{3\ r^2}\right) \dfrac{\zeta}{r}\ \dfrac{(L - L_1)}{2\ l} - \dfrac{1}{2} \left(1 - \dfrac{t^2}{6\ r^2}\right) \left(\dfrac{l}{2\ r}\ \dfrac{r - \eta}{r} - \dfrac{\zeta}{r}\ \dfrac{r - h}{r}\right)}{K_0} \quad \ldots\ldots(83)$$

Taking Equations (69) and (83) for M and H, respectively, and substituting therein $p\,r\,d\,\beta$ for P (uniform water load), and integrating with respect to β:

$$M = p\,r\,(\text{line } G\,C) = p\,r^2\,\frac{L - l}{L} \qquad \text{.............. (84)}$$

$$H = p\,r\left(1 - \frac{t^2}{6\,K_0\,r^2}\right) \qquad \text{..................... (85)}$$

M and H, thus obtained, are identical in effect with the value of H given in Equation (24), thus verifying the case of uniform water pressure as a special case of non-uniform pressure.

Deflection

The arch deflection at a point, the radius of which makes an angle, ψ (Fig. 14(d)), with the center line, is given by the expression,

$$\delta = \frac{H}{E\,I}\int_{\phi_0}^{\psi} z_1\,q\,d\,L - \frac{1}{E\,I}\int z\,s\,d\,L \qquad \text{.............. (86)}$$

in which, the integration of the second term is from ϕ_0 to ψ, when ψ is greater than β, and from ϕ_0 to β when ψ is less than β.

Referring to Fig. 14(b), (c), and (d):

$$q = (\text{line } O\,G) + (\text{line } G\,H) - (\text{line } O\,Q) + (\text{line } B\,F) - (\text{line } F\,E)$$

$$= \frac{r \sin \phi_0}{\phi_0} + \frac{M}{H} - r \cos \phi + \frac{\rho_1^{\,2} \sin \phi}{r \tan (\phi - \psi)} - \frac{\rho^2 \cos \phi}{r} \qquad \text{.......... (87)}$$

$$s = (\text{line } S\,R) + (\text{line } A\,T) + (\text{line } V\,D)$$

$$= r \sin (\phi - \beta) + \frac{\rho_1^{\,2} \cos (\phi - \beta)}{r \tan (\phi - \psi)} + \frac{\rho^2}{r} \sin (\phi - \beta)$$

First, integrating the case of ψ greater than β:

$$\begin{aligned}\delta = -\frac{H\,r^3}{4\,E\,I}\Bigg[&4 \operatorname{vers} (\phi_0 - \psi)\left(\frac{\sin \phi_0}{\phi_0} + \frac{M}{H\,r}\right) \\ &+ \left(1 - \frac{t^2}{6\,r^2}\right)(\cos (2\,\phi_0 - \psi) - \cos \psi) \\ &+ 2\left(1 + \frac{t^2}{3\,r^2}\right)(\phi_0 - \psi) \sin \psi\Bigg] \\ - \frac{r^3}{4\,E\,I}\Bigg[&\left(1 - \frac{t^2}{6\,r^2}\right)(\sin (2\,\phi_0 - \beta - \psi) - \sin (\psi - \beta)) \\ &- 2\left(1 + \frac{t^2}{3\,r^2}\right)(\phi_0 - \psi) \cos (\psi - \beta)\Bigg]\end{aligned} \qquad \text{.............. (88)}$$

For the case when ψ is less than β, the quantity $(\psi - \beta)$ in the second term of Equation (88) becomes $(\beta - \psi)$ and the quantity, $(\phi_0 - \psi)$, becomes $(\phi_0 - \beta)$.

The writer has computed values of $\frac{H}{P}$, $\frac{M}{H\,r}$, and δ, for the subtended angles, 90°, 100°, 110°, and 120°, and for various thicknesses of arch (see Fig. 15). For each subtended angle, the deflection curves have been plotted for three different values of β, namely, $\beta = 0$, 15, and 35 degrees. The coefficients

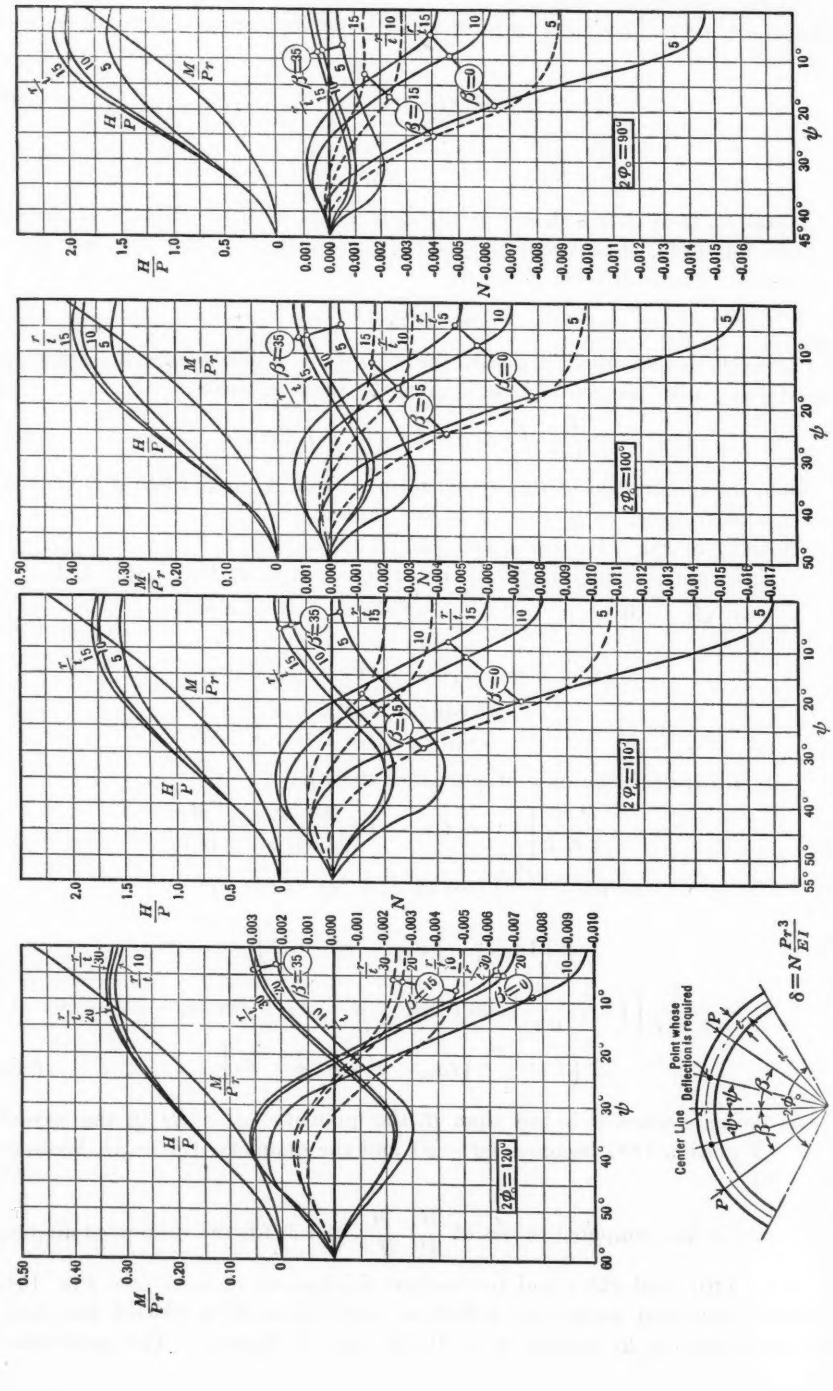

FIG. 15.—INFLUENCE CURVES FOR CIRCULAR ARCHES WITH RADIAL LOADS.

shown on the diagrams are for the imaginary case when P, r, E, and I are each unity, and for any given case the actual deflection is obtained by multiplying the coefficient by $\dfrac{P\,r^3}{E\,I} = \dfrac{12\,P}{E}\left(\dfrac{r}{t}\right)^3$.

The values of $2\,\phi_0$ and $\dfrac{r}{t}$ cover most cases that occur in practice, and deflection coefficients for intermediate values may readily be found by drawing subsidiary interpolation curves.

From the method of deriving the curves (Fig. 15), and by reason of Maxwell's theorem of reciprocity, it will be seen that they are influence curves for the various quantities they represent, and may be used to find these quantities for any conditions of loading; that is, either concentrated loads, uniformly distributed loads, or non-uniformly distributed loads. The last case is the one which is of immediate interest.

Fig. 16 will serve to show the method of finding the moment, thrust, and deflection in a particular case. The data assumed are as follows:

$$r = 100 \text{ ft.}$$
$$t = 6.67 \text{ ft.}$$
$$\frac{r}{t} = 15$$
$$2\,\phi_0 = 90°$$

Water load = 3 000 lb. per sq. ft. at crown, and varying as shown in the load diagram.

$$p = p_e\,\frac{r_u}{r} = 3\,000 \times \frac{103.33}{100} = 3\,100 \text{ lb. per sq. ft.}$$

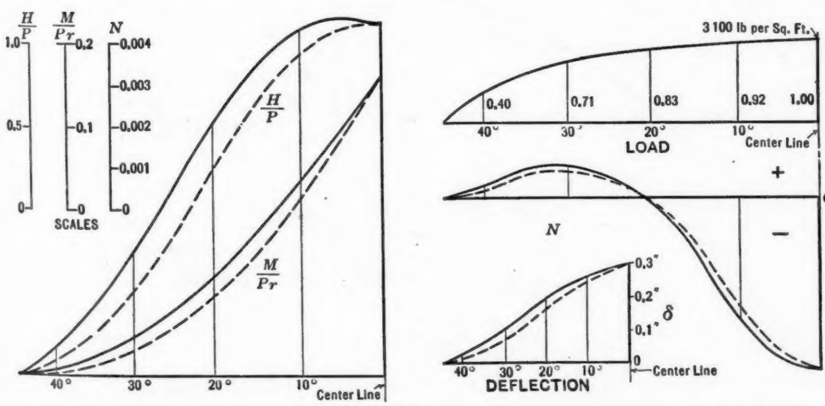

FIG. 16.—METHOD OF FINDING MOMENT, THRUST, AND DEFLECTION IN A GIVEN CASE.

Now, P can be regarded as the concentrated load pressing on the portion of arch subtended by 1 degree. Hence,

$$P = 3\,100 \times \frac{\pi}{180} \times 100 = 5\,410 \text{ lb. at the crown}$$

If P were to remain constant in magnitude, but were to move to various points of the arch, the value of $\frac{M}{H\,r}$, H, and N (for crown deflection), would be read off from the ordinates of the curves shown in full lines corresponding to the position of P. Since P reduces as it moves away from the crown, these ordinates must be reduced in the same manner. Thus is derived the dotted curves, the ordinate at any point of which would give the value of the quantity required for a concentrated radial load acting at that point, and equal in magnitude to the actual load pressing on an arc of 1° at that point. Regarding the actual load as a series of concentrated loads it is seen that the value of the quantity required, under the actual conditions of loading, is obtained by finding the total area under the dotted curves.

In the case shown the areas are: For $\frac{H}{P}$, 48 degree-lb. units; for $\frac{M}{P\,r}$, 5.02 degree-ft-lb. units; and for N, 0.0376 degree-ft. units. Hence,

$$H = 5\,410 \times 48 = 259\,000 \text{ lb.}$$

$$M = 5\,410 \times 100 \times 5.02 = 2\,715\,000 \text{ ft-lb.}$$

and, taking,

$$E = 288\,000\,000 \text{ lb. per sq. ft.}$$

$$\delta = \frac{12 \times 5\,410 \times 15^3 \times 0.0376 \times 12}{288 \times 10^6} = 0.342 \text{ in.}$$

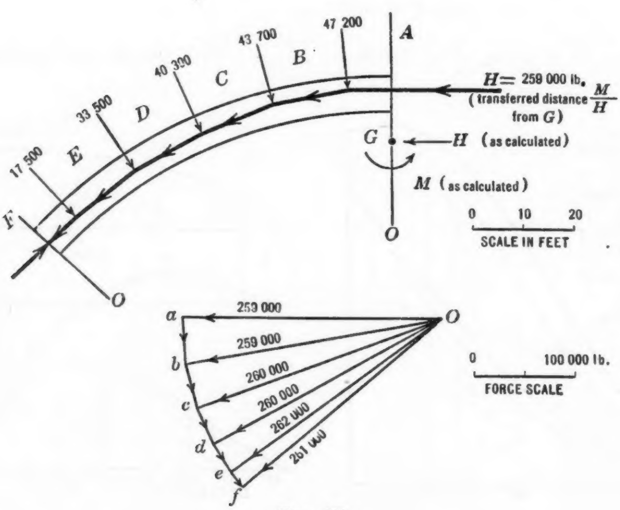

FIG. 17.

The deflection influence lines for $\beta = 15°$ and 35° have not been shown, but the deflections at these points can be found in exactly the same manner (Fig. 16). The developed deflection curve for the arch would be as shown in dotted lines, the corresponding curve for a uniform load of 3 000 lb. per sq. ft.

being shown in full lines. The importance of taking into account the distribution of arch load will be clear from this example.

To determine the stresses in the arch, it is necessary to sketch the conditions of loading as shown in Fig. 17, in which the water load has been replaced by concentrated loads acting on each 9° of arc. It is to be noted that five forces only were taken for the sake of simplicity. In practice, further sub-division is necessary. Now, the moment, M, was introduced to neutralize the moment caused by transferring the line of action of the thrust, H, to pass through the elastic center of gravity, G. The distance from G to the actual line of action of H is, therefore, $\frac{M}{H} = 10.47$ ft. This locates H, and the determination of the resultant force acting at any point can then be effected by the usual procedure. The value and position of the thrust at any point being known, the stresses can be found by the trapezium rule.

APPENDIX III

Notation

The following notation has been used:

a = distance between sections assumed for the purpose of analysis.
d = lever arms of partial water loads (Fig. 18).
d_R = moment arm to crest reaction.
d_z = moment arm to areas on $\frac{M}{I}$-diagram.
δ = deflection.
E = Young's modulus.
f_s = average shear stress, in pounds per square foot.
G = rigidity modulus.
I = moment of inertia.
M = resultant bending moment.
M_R = moment due to crest reaction.
M_w = bending moment due to water load.
p = unit water pressure.
R = crest reaction.
t = arch thickness, in feet.
w = partial water load.
$W = \Sigma w$ = total water load.
z = partial area of $\frac{M}{I}$-diagram regarded as a load.
Z = total area of the $\frac{M}{I}$-diagram regarded as a load $= \Sigma z$.

Arithmetical Computation of Beam Deflections

The following arithmetical method of finding the beam deflection curves is based on the well-known fact that the bending moment diagram of a beam

or cantilever loaded with its $\frac{M}{I}$-diagram, wlll represent, to a suitable scale, the deflection curve. This fact is derived from the elementary theory of beams, which may be summarized in the five expressions:

$$
\begin{aligned}
E\,I\,y'''' &= \text{load for unit length;}\\
E\,I\,y''' &= -\text{ shear;}\\
E\,I\,y'' &= \text{moment;}\\
E\,I\,y' &\ \text{gives the slope; and,}\\
E\,I\,y &\ \text{gives the deflection.}
\end{aligned}
$$

It is obvious from these expressions that the process of repeated integration by which the bending moment is derived from the load diagram will also make it possible to find the deflection from the $\frac{M}{I}$-diagram, using an appropriate scale. This is the basis of Mohr's graphical method. The writer considers that a tabular arithmetical method of doing the same thing is preferable because it eliminates inaccuracies due to errors in drawing and the work can be done very quickly by an intelligent computer, who need not have any knowledge of the theory on which the work depends.

Table 3 gives, in a convenient form, values of I, load, bending moment, etc. Consider the irregular beam, $A\ B\ C\ D$, of unit width (Fig. 18(b)) with a load diagram as shown by $E\,F\,G\,H\,I$, and the reaction, R (Fig. 18(a)). The beam is divided by any convenient number of equi-distant Sections 1, 2, 3, etc., spaced a units apart, and the points of application of the partial loads between the sections are determined, either by the simple geometrical construction or by the simple formula for finding the center of gravity of a trapezium. When the distance between sections is small compared with the total length of the beam, as is generally the case, it may be sufficiently accurate to assume the partial loads to be applied midway between the sections, except for the uppermost section, where the distance is $\frac{a}{3}$ from Section 1.

In tabulating the moments:

$$
\begin{aligned}
M_1 &= w_1 \times d_1\\
M_2 &= w_1\,(a + d_1) + w_2\,d_2\\
&= M_1 + w_1\,a + w_2\,d_2\\
M_3 &= M_2 + (w_1 + w_2)\,a + w_3\,d_3
\end{aligned}
$$

Hence, at any section simply add the progressive moment up to the preceding section; the progressive load multiplied by the constant distance, a, and the moment due to the partial load between the section considered and the preceding section. By making a equal to 20 ft. and assuming that the partial loads act midway between the sections, the arithmetic is reduced to the greatest simplicity.

The bending moment caused by the crest reaction R, is the product of the force and its distance from the section considered. This bending moment is thus simply computed for the various sections and deducted from the "water

TABLE 3.—BENDING MOMENTS AND DEFLECTIONS.

Section.	t, in feet.	I.	p, in pounds.	p (average), in pounds per foot.	a, in feet.	w, in pounds.	$W = \Sigma w$.	d, in feet.	M_w, in foot-pounds (added downward).	Remarks.
0	10	83.4	0							
				300	20	6 000	6 000	6.67	40 000	$= d\,w$
								20.00	120 000	$= d\,W$
1	12	144.0	600							
								10	210 000	$= d\,w$
				1 050	20	21 000	27 000		370 000	= sub-total
								20	540 000	$= d\,W$
2	14	228.0	1 500							
				2 625	20	52 500	79 500	10		
3	20	667.0	3 750	......	....	......	......	......	525 000	$= d\,w$
									1 435 000	= Total

d_R, in feet.	$M_R = R\,d_R$, in foot-pounds,	$M = M_w + M_R$, in foot-pounds.	$\frac{M}{I}$.	z.	$z = \Sigma z$ (added upward).	d_z, in feet.	E (added upward).	Remarks.	δ.
							1 710 200	= Total.	
......	....	....	......	−920	37 260	....	9 200	$= z\,d_z$	0.071
						10	763 600	$= Z\,d_z$	
13.3	− 53 300	− 13 300	−92						
				9 480	38 180	20	955 800	= sub-total.	0.040
						10	94 800	$= z\,d_z$	
33.3	−133 300	+236 700	1 040						
				28 700	28 700	20	574 000	$= Z\,d_z$	0.012
						10	287 000	$= z\,d_z$	
53.3	−213 300	+1 221 700	1 830	......	......	....	....	....	.....

moment", M_w giving the resultant moment, from which $\frac{M}{E\,I}$ is found. These values are then regarded as the ordinates of the new load diagram, and the same process is repeated, in this case starting at the bottom and working upward.

Table 3 shows the method of computing points on the deflection curve for the beam dimensions and loads shown in Fig. 18, only three sections being taken for the sake of simplicity. It will be seen that in this method allowance can very easily be made for any non-uniformity in dimensions or loads. Thus, if the ratio of thickness to radius of curvature is large, the value of I given by Equation (44) may be corrected by the use of the curve given in Fig. 3.

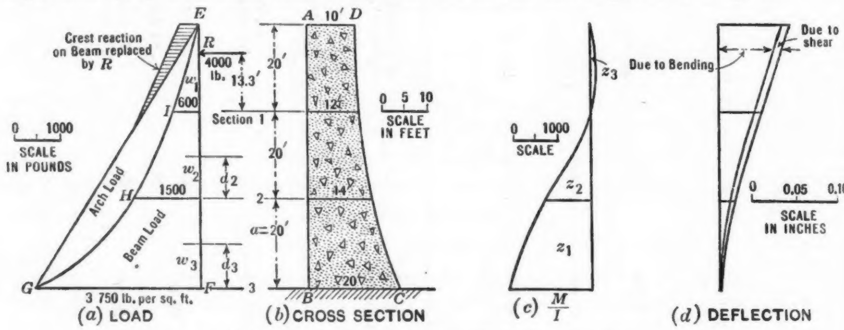

FIG. 18.—BEAM LOADS AND DEFLECTIONS.

Shear deflection is computed separately where necessary, by taking in turn the portions of beam between each pair of sections, and considering each portion subjected to a uniform shear stress equal to the average net load divided by the average thickness. The method of computing is obvious from Table 4. The rigidity modulus, G, is assumed to be 115 000 000 lb. per sq. ft., and the deflection per 20 ft. $= \frac{20 \times 12}{115 \times 10^6} f_s$ in.

TABLE 4.—SHEAR DEFLECTION.

Section.	t, in feet.	Load, in pounds.	Stress, in pounds per square foot.	Mean stress, in pounds per square foot.	Partial deflection, in inches.	Total deflection, in inches.
3	20	75 500	3 780	.....	.....	.....
2	14	23 000	1 640	2 710	0.006	0.006
1	12	2 000	170	905	0.002	0.008
0	10	0	0	85	0.000	0.008

APPENDIX IV

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PAPERS AND DISCUSSIONS

EVALUATION OF WATER RIGHTS*

By John E. Field,† M. Am. Soc. C. E.

Synopsis

The methods and conclusions given in this paper are based on twenty years' experience in making appraisals wherein the value of water rights was an important part. This has involved water values in excess of $10 000 000.

Appraisals on New Projects

Somewhat related to an appraisal is the estimated cost of irrigation works. The question of the tangible, physical, and water values back of the bonds always arises in the event of financing. In the case of a new district, these are largely estimates and generally involve the taking up of new rights. Their value at the time of filing is almost nil, so that the value of a proposed project is generally limited to the physical works, to the rights of way, including interest during construction, and to organization expense, etc.

On some of the projects with which the writer was connected, old water rights were purchased, including old reservoirs, reservoir sites, and old canals. In such cases appraisals were required for the works purchased, particularly for the value of the water itself.

The new works generally destroyed or replaced old ones, the water being used over the entire new system. At the time of purchasing old works, and primarily to obtain the old water rights, whatever amount was paid for those portions of the old works that were abandoned or destroyed, was added to the cost of the water or water right.

Early Appraisals

The writer's early appraisals were based almost wholly on data gathered concerning the actual sale and transfer of water, a rather simple matter.

Note.—Written discussion on this paper will be closed in August, 1928.
* Presented at the meeting of the Irrigation Division, Denver, Colo., July 14, 1927.
† Cons. Engr., Denver, Colo.

Furthermore, it was found that there was a general impression in every locality as to how much water was worth. This influenced the appraisal. These opinions included, not only the value of the water right itself, but of a proportionate interest in the works. On some canals, construction costs are far more than the value of the water itself. On other canals, the physical works are simple and of very low cost. Hence, it must be borne in mind in speaking of the value of water that the physical values of the works and of the water itself must be separated.

Similarity in Values

The market value of an acre-foot of water in a given locality does not vary whether its use be domestic, manufacturing, or agricultural. Further, stored water has little more value than unstored water. A proviso, however, is essential to this last statement, namely, that the use of the unstored water must be economical and beneficial.

Consider in the abstract an acre-foot of water. It may be easily obtainable during the flood season, yet in the absence of other acre-feet, procurable at other times and necessary to bring the crop into the flood period and later carry it to full maturity, that acre-foot of flood water would be without value. If, however, it entered into the production, in whole or in part, of a crop between seeding and harvest, it has a value almost, or quite, equal to the value of an acre-foot used at any period of crop growth. The use of flood waters means, generally, a saving of the stored supply. The law of supply and demand alters the prices, of course; but intrinsically, and on the basis of crop production, the value of flood water is as great as that of water obtainable during the low flow period.

The conclusion as to the uniformity in value of water may be startling to some engineers. Except for the need of water for agriculture in the semi-arid region, there would be enough for domestic needs and for manufacturing. The water used for agriculture can be purchased and converted to a domestic use, it can be condemned and appraised at its agricultural value. It is agriculture, therefore, causing, as it does, a demand in excess of the supply, that gives value to water. It is a more largely consumptive use than either of the other two, although in both manufacturing and domestic supplies, the consumptive use is much greater than is generally believed.

The writer has investigated consumptive use by the City of Denver, Colo., and has concluded that during the summer months about 66% of the gross diversions was returned, while in the winter the quantity was 80 per cent. As regards the consumptive use of water for manufacturing purposes, principally for cooling, about 10% of the supply was lost in the transit through the power houses and mills. The idea that practically all the domestic and manufacturing supply returns to the river is just one of those dreams that does not come true.

The value of water for domestic purposes should not be based on the city's necessities, nor on what the city could afford to pay, but rather on the value of water for agricultural purposes. As to manufacturing, the large

intrinsic value lies in the water used for condensation purposes. Large generating plants are often forced to go to localities where water is available rather than to build on sites in all respects preferable, except that the agricultural interferes with the manufacturing use. If the water is used for the direct generation of power the value could be determined by comparison with the cost by steam or the use of electric power.

The statement that an acre-foot of water, whether stored or direct flow, varies little in value, is not so obvious, as the first conclusion. It is true that an acre-foot of water in the latter part of the season is much more valuable than in the early and flood periods, and that an available quantity of water during the time of very low flow is much more valuable than during the flood period. This is confirmed by the variations of value of water having different dates of decree.

Obviously, flood rights are of considerable intrinsic value. They permit a greater area to be cultivated, encourage the growing of early maturing crops, and stimulate a more general rotation of crops than would otherwise be practiced. Reservoirs built for the storage of water are generally much more expensive per acre-foot of capacity than are canals. Deducting the cost of structures from the value of water plus structures would show that water values are more nearly equal than the values of structures plus water. In the two types the cost of a dam with its accessories often approaches the value of water plus structures even after it is fully used and is a successful and going concern. Thus, waters having decrees for storage purposes and for direct use have values varying with the dates of decree and independent of the method or character of works used in securing it.

In his early experience in appraising water rights, it seemed to the writer to be simply a matter of ascertaining the sale value of the water as determined by a number of sales under comparable conditions and in the same locality. The use of such data, however, led to the most irreconcilable differences as well as to absurd results. It early became apparent that there was a vast difference in value between water "direct-flow" as compared with "stored water". In some cases, the opposition engineers insisted on converting one into the other, and basing the value on the quantity theoretically procurable during the entire irrigation season. While such a method appeared ridiculous at that time—and indeed, the results were ridiculous—yet it now appears that it is, in part, correct. Due regard, however, must be given to the quantity actually produced by each and to the date of the decree. To judge stored water by comparing it with value of water diverted under old decrees, or *vice versa* although the time of use and application were the same, would be wrong. The cost of structures in each case added to the water value gives the value generally quoted.

Apparent and True Values

To those familiar with the transfer of water in Colorado from one ditch to another, the purchase price is not a true measure of the value. In most cases there are conditions and provisions in the agreement which very ma-

terially alter the given value. The purchase of 10 sec-ft. of water at \$3 000 per sec-ft. is \$30 000, but when to this figure are added the Court costs of several thousand dollars, and the cutting down of the quantity allowed to be transferred, the rate might be, and usually is, more than doubled. Where water rights are purchased and transferred, it is almost invariably true that the seller is anxious to sell and the purchaser not compelled to buy; that whatever that seller gets for his surplus water is so much net money to him. Too frequently, he is selling something he has never used or, at best, something that he at one time used but through changed conditions, due to seepage returns, etc., he no longer needs.

The Method Proposed

In an effort to reconcile these differences, the writer sought a more rational method for the determination of the value of a water right. In one case, he selected three of the larger, more substantial, and well-known ditches on the Arkansas River—ditches on which not only were the values of the structures known, but for which the daily flow into the canals over a period of at least eight years was of record. As each of the three canals has a number of priorities of different dates, varying not only as to their own rights, but as to those of the other two ditches, the appraiser was able to construct curves showing value and priority dates.

In this case about 200 priorities and more than 100 diversions were to be appraised. The uses were: Domestic, the year round; manufacturing, some used intermittently and some continuously; and, agricultural, where the irrigation season varied from 60 to 300 days in the year. Also, the records for the domestic and some of the manufacturing uses were expressed in gallons per year without reference to the maximum flow or the hours or days when the pumps were in operation. Therefore, it was necessary to develop curves showing the value of different priorities and the percentage of their efficiency as compared with a 365-day use; and to use total volumes diverted during the year expressed in acre-feet rather than the irrigation season and the rate of flow expressed in cubic feet per second.

It was decided, therefore, to use as the maximum possible, the theoretical quantity that could be diverted under the decree, and to determine the percentage of efficiency or delivery by comparing the actual delivery with this theoretical maximum.

First, the selling price of a share in the ditch company was ascertained. For each ditch the price had been quite uniform over a period of four or five years, and several transfers of comparatively large blocks of stock had been made. Furthermore, whether the stock was sold in quantity or in individual shares, the price varied little. The value of the works, including the water rights, was determined by multiplying the average sale price by the number of shares. From the books of the companies and from personal investigations on the ground, the writer determined the replacement value of the structures, making due allowance for the fact that some parts, embankments in particular, were more substantial and valuable than when they

were first constructed, having passed safely through the most dangerous period of their existence, namely, the first few years. Having determined the structural and going-concern value, as well as the full value of the works, the difference between the value of the stock and the value of such works was called the value of the water rights.

In reality, the three-ditch systems are mutual ditch companies in which the water users own non-interest bearing shares roughly proportioned to the area irrigated, or to the necessity of the users. Generally speaking, stock is sold only to users, and the proceeds are used for the payment of debts, for improvement, or for the reduction of assessments. The ownership of shares without land on which to use the water would simply mean annual assessments without benefit.

Next, the quantity of water carried by the three canals was ascertained, over a period of eight years. A longer period would have been desirable, but the records were not uniform beyond that period; indeed, many were missing, so that, perforce, study was limited to eight years. Having ascertained the average number of acre-feet of water diverted per annum, the value per acre-foot was determined by dividing the water-right value by the number of acre-feet diverted. An example, taken from the report, will show clearly the method used:

"FORT LYON CANAL

"Priorities are April 15, 1884 for 164.64 cu. ft.
March 1, 1887 " 597.15 " "
1893 " 171.20 " " "

The value of this system is, according to its President, $6 000 000, and that of the structures about $2 000 000, leaving the water value as $4 000 000, with $600 000 of indebtedness, a total value of $4 600 000.

A careful analysis of the daily records for eight years shows the flow credited to the respective priorities to be as follows:

From reservoirs	47 000	acre-ft.
Average for the 1884 decree	80 000	" "
" " " 1887 "	90 000	" "
" " " 1893 "	8 300	" "
Total	225 300	acre-ft.

The value of the water per acre-foot is, therefore....... $20.42

Reservoir water = 47 000 × $20.42 = $960 000.

Decree 1884, therefore, is worth $1 635 000 = $9 925 per cu. ft. per sec.
" 1887 " " " 1 838 000 = 3 080 " " " " "
" 1892 " " " 169 000 = 990 " " " " "

The 1884 decree delivers 80 000 acre-ft. out of a possible 119 200, for 365 days during the year. Its percentage of efficiency is, therefore, 67.15. Therefore, $9 925 represents 67.15% of the maximum value; that is, 100% would be worth $14 900 per cu. ft. per sec. Similarly, the other priorities give the same result: The 1887 decrees have an efficiency of 21%, and the 1893 decrees

have an efficiency of 6.7%, for this particular canal. In case the value of the physical works was placed too high, the value of the priorities would, of course, be greater than that shown.

In ascertaining the number of acre-feet delivered under each of the decrees on the canals, the daily records were studied, and whenever only the oldest decree was being filled or partly filled, the entire quantity was assigned to the first decree. When a quantity was shown in excess of the first decree, but not greater than the first and second combined, the remainder was assigned to the second decree; whenever the quantity carried was in excess of the two first decrees, the excess was credited to the third decree; and so on through all the decrees. The result of this study was that the writer had the average number of acre-feet delivered per annum by each decree, and multiplying that quantity by the acre-foot value as determined gave the value of the decree. Dividing this by the quantity of the decree, in second-feet, gave the value per cubic foot per second. As the values were to be used in comparison with supplies for domestic and manufacturing power and irrigation, and with stored and unstored water, it appeared necessary to have some common basis for comparison. The quantities diverted under each priority varied in amount from day to day and from year to year. The only common ground seemed to be what would have been the quantity diverted had every priority run for the entire year.

Grouping Priorities

Having ascertained by this method the value of ten or twelve priorities on the river, it is possible to construct a curve that can be used in determining values of other decrees of other canals. In the case used as an illustration it was shown that all priorities having a date earlier than 1874 were of equal value. That this was true was demonstrated by plotting all the priorities as to date and quantity, the quantities being the aggregate of the water decreed prior to a certain date. In this way, it was ascertained that even in years of minimum flow the supply in the river was in excess of all the decrees of 1874 or earlier. It was found, also, that priorities could be grouped. These groups were separated by some decree calling for a very large quantity of water. Thus, the application of the data and the determination of the value of specific water rights were much simplified.

In Fig. 1 the curve to the left relates in part to the three canals investigated and to their actual value as determined, while the several curves to the right represent values as determined from the sale of water to be transferred, illustrating the impracticability of using that method for the determination of real values. These latter curves illustrate, also, the gradual increase in values of water rights over a period of years.

Starting at zero on the year of appropriation, they rise uniformly to the time when the first information is available, about 1897; from that point the actual sale prices indicate quite a uniform advance in values, a sudden rise during the World War period, with no rise, possibly a decrease, subsequent to about 1921. The steepness of the curve between 1900 and 1905 is prob-

ably due to the effect of the Reclamation Act, while the break in the curve in 1907 is probably due to the financial panic of that year. However, one is not justified in using this curve as anything other than illustrative.

It should be clearly understood that this method is suggested for the purpose of laying a foundation only, for detail study. On each stream the value must be determined independently of that on any other system, but even after having obtained a basis for further detailed work, it should be used more as a guide and to permit the construction of curves where many data are missing.

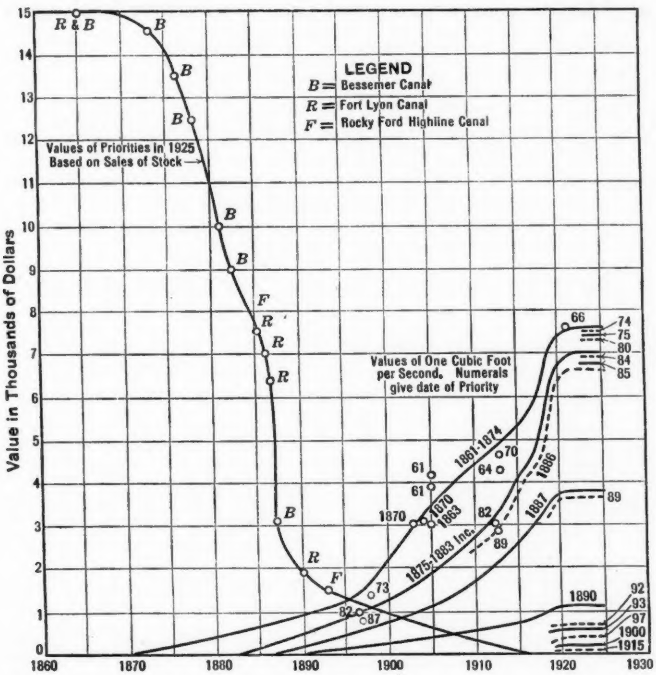

FIG. 1.

The law of supply and demand rules, and it often happens that under an old priority, where there is an abundance of water, the value of the water is less than under a later priority, where the abundance of land makes a marked demand. The matter of crops, productivity of the soil, the markets, all these must be taken into consideration by the appraiser in his detailed study. In arid and semi-arid regions the value of the water is its value for its most important use, namely, for irrigation. Therefore, the appraiser may determine from curves based on agricultural use the value of the domestic water rights for cities, and the value of water for railroad purposes. In the latter case, however, it often happens that the value of the water is the cost of bringing in some substitute which might involve the hauling for long distances. Where water is as scarce as this, however, a value could hardly be established under the method suggested.

Colorado is fortunate in having a law which requires the installation of automatic registering devices on all canals. Since that law was passed the State Engineer has exercised diligence in having automatic registers placed throughout the State. When the records from these are matured and available, it will be an easy matter to ascertain the stock value of an irrigation system and the average number of acre-feet diverted per annum and from this to deduce the value of an acre-foot of water. It will involve no great amount of labor to place the proper values on water rights. As has been stated, this can be done safely only by those who are familiar with the particular locality or willing to devote a large amount of time to ascertain local conditions.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

ADMINISTRATIVE WATER PROBLEMS

A SYMPOSIUM*

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TRANSMISSION AND DELIVERY OF RESERVOIR WATER

By G. Clyde Baldwin,* M. Am. Soc. C. E.

Throughout the early stages of irrigation development in the West reservoirs were of little consequence. Land was watered by direct diversion from the natural streams with small effort and at little expense. The success of the pioneers naturally attracted others; more difficult enterprises were undertaken; and following the enactment of the Federal Reclamation and Carey Acts, really large-scale undertakings were initiated. By this time, also, the low-water flow of many of the streams had proved to be insufficient to supply all demands. For this reason, and because of the feasibility, under these laws, of financing the construction of dams and other necessary works of considerable magnitude, storage reservoirs began to play a part of constantly increasing importance in all reclamation plans; until, to-day, they are usually considered to be the foundations upon which most new irrigation projects must be based.

Low unit storage cost, together with the physical possibility of making gravity deliveries, chiefly governed the location of these early reservoirs. Scant consideration was given to the other difficulties which might later be experienced in connection with their operation.

Where the selected impounding basin is located close to the irrigable lands, on otherwise dry channels, or on small streams where no conflict with other water rights exists, these operation or delivery problems are comparatively unimportant. They become very real and worthy of consideration, however, where the storage must be transmitted for long distances down the natural channel of a large stream, most of the normal flow of which has been awarded to other and earlier appropriators.

Situations of this character are now to be found in most of the Western States, but probably none is more complicated or involves the administration of such large quantities of water as that of the Upper Snake River Basin in Idaho and Wyoming. In that portion of this area which, for water distribution purposes, is administered as a single unit, four major and several minor storage reservoirs are in operation. Without taking into consideration small diversions on the upper reaches of the streams, which are relatively isolated and where net consumption of water is usually small, water is diverted from the river and its larger tributaries through about 120 principal canals.

The four large reservoirs are Jackson Lake, situated at the upper end of Jackson Hole, Wyoming, and only a few miles south of Yellowstone National Park, with a capacity of 847 000 acre-ft.; Henrys Lake, near the head-waters of the North or Henrys Fork of Snake River, the capacity of which is about 75 000 acre-ft.; American Falls, on the river just above the Town of American Falls, Idaho, where the full capacity of 1 700 000 acre-ft. is available for use

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this year (1927), the first since the completion of the dam; and Lake Walcott, created by the Minidoka diversion and power dam of the U. S. Bureau of Reclamation, where about 107 000 acre-ft. are normally impounded and can be used, at the sacrifice of power head, in times of urgent need.

The canals range in size from those with a capacity of only 1 or 2 sec-ft. up to one of more than 3 600 sec-ft. The maximum for the entire group amounts to nearly 30 000 sec-ft., while the total diversions from May 1 to September 30 in a normal water supply year equal almost 6 000 000 acre-ft.

Henrys Lake Reservoir is owned and utilized entirely by canals in the vicinity of St. Anthony, Idaho, while Lake Walcott belongs to the adjacent Minidoka Project. Roughly, 15% of the combined storage impounded in Jackson Lake and American Falls Reservoirs belongs to canals in the so-called Idaho Falls Section; 14% to the Minidoka Project; 38% reserved for as yet undeveloped enterprises; 2% to the Idaho Power Company; and the remainder, or 31%, to the Twin Falls-Jerome Canals which divert at Milner Dam, the extreme lower end of State Water District No. 36.

The natural stream channels must be used for carrying all stored water, except that impounded in Lake Walcott, from the reservoirs to the points of final diversion and use. In the case of Jackson Lake water, this means transmission of more than one-half the total volume for a distance of about 300 miles, past the head-gates of the Idaho Falls region canals, and on through the American Falls and Lake Walcott Reservoirs.

Normal flow water rights have been decreed at the canal head-gates, but stored water, on the other hand, is acquired at the reservoir. Both the States of Wyoming and Idaho provide by statute for the use of stream beds as channels for the conveyance of stored water. However, there is no prescribed basis for converting a reservoir right into a right at the point of diversion many miles down stream. Every one agrees that some deductions should be made for transmission loss, but there is a wide divergence in views as to how this loss shall be determined. This ranges from a minimum proportional charge based on estimated additional evaporation and seepage resulting, respectively, from the extra exposed surface and wetted perimeter caused by the storage run, up to a direct proportional quantity tax through each small section of river channel where net losses are known to occur, with no corresponding credits in sections where the river shows a net gain.

During the first few years after the completion of the original temporary dam at the outlet of Jackson Lake, attempts were made to deliver the storage to the lower valley in a series of flush heads. Prior to the arrival of each of these flushes, canals in the Idaho Falls area were staked and river riders were employed to make certain that this normal draft was not exceeded during the storage run. Under this delivery plan Lake Walcott was operated as a catch-basin and equalizing reservoir. This method resulted in net storage losses amounting to between 25 and 30%, while the alternate high and low river stages were very destructive to the less permanent diversion dams along the stream and made difficult the maintenance of any uniform flow in the canals. It was, consequently, unsatisfactory alike to stored water and normal flow users. Beginning in 1912 this method was abandoned in favor of a continuous

flow system which, with certain modifications pertaining chiefly to transmission losses, has been in use since that time.

Under the latter system the rate at which storage is released from the reservoir varies with the needs of the stored water users, but abrupt fluctuations of any magnitude are avoided. Allowance is made for time ot transmission and loss in transit; then the intervening canals having normal flow rights are regulated in accordance with the decrees to insure proper deliveries each day. The daily determination and frequent regulation of flow in each canal, as well as the maintenance of continuous records at numerous river points, are required.

For many years, and to a considerable extent even now, the amount to be charged against this storage as transmission loss continued to be subject to dispute and was provocative of much dissension as to stored and normal flow, or on geographical lines, between lower and upper valley water users.

Numerous partial investigations of different phases of this problem were authorized from time to time. Most of these, because of the experienced and disinterested personnel obtainable, were conducted by, or under, the supervision of the Water Resources Branch of the U. S. Geological Survey. These investigations served gradually to accumulate certain facts concerning river losses and gains and the reasons therefor. The problem is an extremely complicated one, however, involving many variable quantities, so that even if much more money were expended and the investigation made comprehensive over a period of years there appears to be some doubt as to whether a transmission loss schedule can be evolved, which will be absolutely equitable under all conditions.

Meanwhile, deliveries have been effected from year to year under what might be termed compromise schedules, based partly on investigation or fact and partly on mutual concession by the different classes or groups of users. Actual applied losses from Jackson Lake to Blackfoot have ranged from 5% in some of the earlier years to about 15%, with the average for recent years about 13 per cent. These are subdivided as follows: From the outlet of Jackson Lake to Heise, Idaho (canyon section), 2½%; from Heise to Lorenzo (so-called Snake River cone), actual percentage loss each day as determined from the average of the two preceding days' records, with a minimum charge of 3%; from Lorenzo to Woodville (section where a net gain in the river is usually noted), ½%; from Woodville to Blackfoot (heavy loss section), 6 per cent.

Another troublesome item is the equitable segregation of stored water from normal flow at the outlets of the respective reservoirs. For Jackson and Henrys Lakes the practice of relying chiefly on the reservoir capacity tables for the determination of the amount of storage released daily, has been followed for a number of years. Special investigations indicated that while this method does not accurately apportion all the factors that make up the total outflow at all times and under all conditions, it is, in general, a reasonably fair plan of distribution and has the advantage of being easily operated. This method automatically credits the normal flow with all increments from bank

storage return in lieu of the old natural lake storage and the extra water-surface evaporation.

The situation at American Falls is somewhat different. Here, there was no initial lake prior to the construction of the reservoir and little is known as yet concerning the relative values of such items as deep percolation loss and bank storage return, while the stored water will be subject to an evaporation loss charge for practically the entire exposed water surface. This reservoir is situated in a section where the river previously showed a net gain from large springs and ground-water sources. Data obtained by the U. S. Bureau of Reclamation during the summers of 1924, 1925, and 1926, suffice to determine that a little more than 50% of this gain comes from surface streams which may be gauged even under full reservoir conditions and that the total inflow or gain through the reservoir is quite consistent in its fluctuations with the changes noted in the measurable half.

The normal flow which should pass the outlet gates can be approximated, therefore, by adding to the water which enters the reservoir through the main river channel the product of the daily measured gain and the predetermined coefficient. This method is both laborious and expensive because it requires daily measurement and summation of the flow of about twenty-five rather inaccessible streams. Furthermore, there is no assurance that a coefficient which was applicable in one year will also apply in another; but it affords a determination that is independent of evaporation, percolation, bank storage return, and other such items, all of which were studied during the summer of 1927 with the idea of evolving a simpler procedure for the future.

The use of space in Jackson Lake Reservoir for the storage of decreed water rights and the later release and delivery of the water thus held back were authorized by mutual consent of all water users in 1924 and again on a more restricted basis in 1926. This plan encouraged rotational use of water and undoubtedly helped to promote higher duty usage during those years when the total supply was deficient. Its adoption adds many complications to the work of water administration and because of the departure from normal procedure the plan is perhaps subject to attack on the question of damage.

The human relations feature of the water distribution work is probably more important than any other phase. It is often said that in an irrigated country an otherwise upright man will not hesitate in times of drouth to steal water and that the most peaceable citizen under ordinary conditions may quickly become one of the most violent if he thinks his water rights are being attacked or infringed in any way. Certainly, water disputes are easily started and sometimes become very bitter.

In this respect the Upper Snake area is not materially different from other sections; but there appears to be a growing tendency to try to adjust these disputes without violence and without recourse to long drawn out litigation. This attitude is believed to be largely attributable to a so-called "Committee of Nine" the members of which are elected each year by the water users to act as advisers to the district water master. At any rate this Committee has been most helpful in making contacts between the administrative officer and the individual water users. It passes on the annual distribution expense budget

and on all matters of policy that are not clearly defined or established. In many other respects it may be compared to a Board of Directors for the Water District, of which the Water Master, or Deputy Commissioner of Reclamation, is the General Manager.

The writer has held the latter position, also that of Special Deputy in charge of stored water delivery, since May, 1919. In addition he has acted as District Engineer for the U. S. Geological Survey in charge of hydrometric work within the same area. In March, 1923, and, subsequently, each year he has also been elected to the position of Water Master, thus further centralizing the water distribution administration. A skeleton organization consisting, in addition, of two engineer assistants and a clerk, is now maintained throughout the entire year while deputy water masters, hydrographers, and other employees are added as needed during the summer months. A limited edition, special report of each year's operations is prepared during the winter while stream-flow data are also made available through U. S. Geological Survey Water Supply Papers and the biennial reports of the State Commissioner of Reclamation.

While of necessity details have been largely omitted in this outline and many important phases of the work have been touched only briefly, it is sincerely hoped that this procedure, which has stood the test of many years actual operation, may be of value in other sections where similar problems are either now being encountered, or where they may develop with more complete water usage.

ADMINISTRATION OF STREAM FLOW

By Edward Hyatt,* M. Am. Soc. C. E.

Synopsis

In this discussion of the administration of water supplies under State control in the seventeen irrigation States, special reference is made to California. The conflict between the opposing legal principles of riparian and appropriative water rights has retarded and penalized water development in California. Efforts are under way to ameliorate this condition.

Comparison of Water Codes

In seventeen Western States irrigation is generally practiced. These include Texas, Oklahoma, Kansas, Nebraska, North Dakota, South Dakota, and all States west thereof. Of these, all but Kansas, South Dakota, and Montana have adopted water codes providing for State administration of the distribution of the waters of their stream systems through the agency of water masters or water commissioners. While the fourteen codes adopted show considerable variance in nomenclature and in points of procedure an analysis reveals substantially the same underlying purpose in each, which is to make possible effective State control of the distribution of water.

Each code provides for the division of the State into water districts. In Colorado and Texas this division has been arbitrarily made by legislative enactment and, in all other States, the water districts are created as needed by the State Water Commissioner, State Engineer, or the official in charge of water matters. In three States the water masters are appointed by the Governor; in one State the water users elect the water masters; and in all others they are appointed by the chief water official. In eleven States the water master has the power of arrest and, in ten, interference with his acts constitutes a misdemeanor. Twelve of the fourteen codes give the water user the right of appeal from the actions of the water master; seven to the Superior, District, or Circuit Court according to the judicial structure of the States, and five to the State office in charge. In three States the compensation of water masters is fixed by the Legislature, in one by the water users, in one by the county commissioner, and in the remaining nine by the State office.

Financing the work of distribution is accomplished in various ways. In one State—Nebraska—it is paid from general State funds and, in five others, from general county funds. In 1927 the California Legislature passed an act by which half the cost will be met by the State and half by the water users. In the remaining seven States the charges are met in different ways, but in each case are later assessed back against the actual water users. The basis of apportionment of cost among the water users is,

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in most cases, the respective quantities of water delivered. In one State—Nevada—it is on the basis of the established water rights.

The methods of administration for the various States are listed in Table 1.

California Procedure

According to the 1920 Census, the irrigated area in California exceeded that in any other State and was more than double that of any other State except Colorado and Idaho. With the statutory water codes approximately the same in all the States and with the greatest irrigated area in California, it would naturally be expected that California would be found in the lead, or nearly so, in the matter of efficient handling of its water resources, particularly in the matter of distribution. This, however, is far from being the case. In both adjudication and distribution California is far behind such States as Colorado, Wyoming, Idaho, and others. The reasons for this condition are not far to seek. The principal obstacle lies in the recognition by California of the riparian principle.

Riparian Rights

Apparently, most of the inter-mountain States have escaped the riparian rights doctrine and its consequences entirely. This rule has caused no end of trouble, and has been a great handicap to California. Years of effort and millions of dollars that should have been put into development have been expended in conflict and litigation. A general uncertainty has been thrown about the legality of water rights, which has added greatly to the difficulties in the way of water projects.

A riparian right rests on land ownership and inheres in land bordering on a stream of water. The right as defined by the California Courts does not depend on use nor cease with disuse; it is superior to an appropriative right except in special instances and is not limited to reasonable use as against the appropriator. It is contrary both in theory and practice to the doctrine of appropriation of water, which rests entirely on use and is, of course, opposed to any general water conservation measures such as have to be resorted to in the West.

Historical

California's water law history has been unfortunate but interesting. The appropriative doctrine was established in the "Fifties" by the miners as a necessity of the time and has been recognized by the Courts from that time on, although not until 1913 was it clearly set forth by statute. The riparian theory was adopted by reference when the common law of England was made the rule of decision in the State by statute of 1850.

The fact that there were two doctrines of water law, contrary in principle, naturally led to many disputes. Neither being adequately set forth by statute such disputes could be settled only by Court decisions. Therefore, in sixty years, about seven hundred State Supreme and Appellate Court decisions on water matters have been given. Covering a range of climatic conditions from humid to arid and such a great length of time, the deci-

TABLE 1.—ADMINISTATION OF STREAM SYSTEMS BY WATER MASTERS, AS OF JULY 1, 1926.

State.	Divided into water districts by:	Water masters appointed by:	Water masters supervised by:	Water master has power to arrest.	Penalty for interference with regulation by water master.	Water users have right of appeal to:
Arizona	State Water Commissioner.	State Water Commissioner.	State Water Commissioner.	Yes.	Misdemeanor.	Superior Court.
California	Chief of Division of Water Rights.	Chief of Division of Water Rights.	Chief of Division of Water Rights.	Yes.	Misdemeanor.	Superior Court.
Colorado	Legislature (State divided into 70 Water Districts.)	Governor.	State Engineer (through Division Engineers).	Yes.	Misdemeanor.	State Engineer (through Division Engineers).
Idaho	Department of Reclamation.	Elected by water users.	Department of Reclamation.	Yes.	Misdemeanor.	Department of Reclamation.
Nebraska	State Department of Public Works.	State Department of Public Works.	State Department of Public Works.	No provision.	No provision.	No provision.
Nevada	State Engineer.	Governor,	State Engineer.	Yes.	Misdemeanor.	No provision.
New Mexico	State Engineer.	State Engineer.	State Engineer.	No provision.	No provision.	State Engineer.
North Dakota	State Engineer.	State Engineer.	State Engineer.	Yes.	Misdemeanor.	State Engineer.
Oklahoma	State Engineer.	State Engineer.	State Engineer.	Yes.	Misdemeanor.	State Engineer.
Oregon	State Engineer.	State Engineer.	State Engineer.	Yes.	Misdemeanor.	Circuit Court.
Texas	Legislature (each county a water district).	State Board of Water Engineers.	State Board of Water Engineers.	No provision.	No provision.	District Court.
Utah	State Engineer.	State Engineer.	State Engineer.	Yes.	No provision.	District Court.
Washington	State Supervisor of Hydraulics.	State Supervisor of Hydraulics.	State Supervisor of Hydraulics.	Yes.	Misdemeanor.	Superior Court.
Wyoming	State Board of Control	Governor.	State Board of Control.	Yes.	Misdemeanor.	District Court.

TABLE 1.—(*Continued.*)

State.	Compensation of water masters fixed by:	Water masters paid by:	Funds from which water masters paid.	Method of collection of funds.	Basis of apportionment of cost among water users.
Arizona	State Water Commissioner.	Water users.	Assessments ordered by Superior Court each month.	No provision.	Discretionary with Superior Court.
California	No provisions.	No provision.	No provision.	No provision.	No provision.
Colorado	Legislature.	County.	General County fund.	General County taxes.	Not apportioned.
Idaho	Water users.	County.	Current County expense fund.	Special County taxes against lands served.	Quantity of water delivered.
Nebraska	Legislature.	State.	General funds of Department Public Works.	General State taxes.	Not apportioned.
Nevada	State Engineer.	County.	County Water District funds.	Special County taxes against lands served.	Established water rights.
New Mexico	State Engineer.	County.	General County fund.	Special Connty taxes against lands served.	Quantity of water delivered.
North Dakota	State Engineer.	County.	General County fund.	Special County taxes against lands served.	Quantity of water delivered.
Oklahoma	State Engineer.	County.	General County fund.	Special County taxes against lands served.	Quantity of water delivered.
Oregon	State Engineer.	County.	General County fund.	General County taxes.	Not apportioned.
Texas	County Commissioner.	County.	General County fund.	General County taxes.	Not apportioned.
Utah	State Engineer.	State Engineer.	Specified Water Master fund.	Assessments against water users by State Engineers.	Discretionary with State Engineer.
Washington	State Supervisor of Hydraulics.	County.	General County funds.	General County taxes.	Not apportioned.
Wyoming	Legislature.	County.	General County funds.	General County taxes.	Not apportioned.

sions are naturally overlapping and conflicting; however, they are in the main not subject of change by legislation, and to a considerable degree constitute the water law of the State. It is of interest that the other irrigation States likewise adopted the common law of England, but that their Courts nearly all followed the lead of Colorado and abrogated the common law provision on the general ground that it was unsuited to the needs of the State. If the California Courts had adopted this same attitude the State would have avoided this difficulty.

California Water Code

On account of conflict between the riparian and appropriative factions it was impossible for many years to enact even an appropriative code, but in 1913 this was finally forced through for the initiation of appropriative rights. In 1917 an adjudication procedure was added and in 1921 the distribution feature. The distribution statute was incomplete in that compensation for water masters was not covered and it was not until the 1927 Session of the California Legislature that the distribution procedure was made workable by the addition of a compensation provision. This law took effect about August 1, 1927.

Adjudication Methods

Under the 1917 amendment two methods of adjudication of water rights were provided: First, a complete procedure for the adjudication of appropriative rights patterned after the Oregon Act, which was thought to be the best then in existence; and the second authorizing judges of Superior Courts to transfer water litigation to the Division of Water Rights for investigation as referee. The results under these two modes have not been as anticipated. The adjudication procedure provided in the Act is a complete and admirable one and effective in a case involving only appropriative rights; but when the *bete noire* of riparian rights enters the picture the situation is changed. This method cannot determine riparian rights, and, as any adjudication should, to be satisfactory, take account of all rights, it is apparent that this type of action will fail where riparian rights of magnitude exist.

To its sorrow California has found that they exist almost everywhere. A good attorney can develop for a client riparian rights which the client himself would never have thought of claiming. Mainly on this account in the last few years most of the adjudications have been handled through the Court reference method. To accomplish this a suit is filed, sometimes a friendly suit, and then by stipulation the case is referred to the Division of Water Rights. In this way rights of all classes, appropriative, riparian, and prescriptive, are included, as the Court has jurisdiction over all such and the Division of Water Rights in acting as the Court's referee can properly take account of all. After making its investigations the Division reports back to the Court, often presents a suggested form of decree, and the desired result, that is, a workable adjudication of rights, is accomplished.

This method is not without its difficulties, however, as it is usually necessary to re-frame the pleadings to the suit, so that all parties cross com-

plain, and to have the Court order into the case all users of water from the stream in question, not already parties to the suit. Therefore, unless the judge and the various attorneys are actively interested in seeing that a true adjudication results, some difficulty is apt to be encountered.

Adjudication Results

Adjudication proceedings on twenty streams in California have been undertaken by the Division of Water Rights, fifteen under Court reference and five under the adjudication procedure provided in the statute. Nine have been completed and eleven are pending. While three stream systems of considerable magnitude, the Stanislaus, Shasta, and Whitewater Rivers, are being adjudicated, the other proceedings have been principally on smaller systems, mainly in the northern part of the State. The increase in activity under the Court reference method has been marked in the last few years.

Distribution

The first instance of distribution of water under the provisions of the 1921 Statute occurred in 1922 when the water users on West Carson River, an interstate stream which flows into Nevada, petitioned for and received service in conformity with that law. Since then, as additional adjudications have been completed, the number of water masters has grown to six, besides some assistants. No method of compensation being provided for it has been necessary for the water users to subscribe voluntarily the greater part of the required funds, the State lending some aid. This method is of course unsatisfactory and has retarded the extension of water master service. As already indicated, this difficulty has now been removed with the adoption of what is believed to be an equitable method of payment for the water masters.

In some instances the necessity for a responsible official in charge of distribution has been so great that agreements have been effected under which a water master could operate. In several cases adjudications were under way but not completed; and in others the complications of the situation were so great that adjudications by any method were deemed practical impossibilities and mutual agreements were reached among the ditch owners which permitted distribution.

The outstanding example in this latter class is Kings River, the most important irrigating stream in California. Approximately 700 000 acres are irrigated with the waters of Kings River and the direct flow rights and diversion capacities amount to 10 000 sec-ft., a flow which obtains in the ordinary season for a very few weeks and which is not reached in a dry season. About 150 000 people, practically all dependent on the results of agricultural pursuits, live in the area supplied by the waters of Kings River, showing the importance of the matter.

This river is an excellent example of the confusion caused by the conflict between the two principles of water rights. More than 200 lawsuits had been filed affecting the water rights of Kings River and more than 40 actual Court decrees entered, which, however, were so complicated and im-

possible of reconciliation that they could not be used as a basis for either adjudication or distribution. After several years of investigation a schedule devised along the lines of past use was suggested by the Division of Water Rights for Kings River and was accepted temporarily by all parties. Under this schedule the water has been successfully distributed for a number of years.

A somewhat similar situation exists in Sacramento Valley. Here as an additional problem the low-lying lands around San Francisco Bay are endangered by salt water entering from the sea. Formerly, the flow of Sacramento River was sufficient to keep the salt water back, but with the development of agriculture in the Upper Sacramento Valley this is no longer the case. A water master with limited authority is in charge of Sacramento Valley.

Speaking generally, both adjudication and distribution are in their infancy in California. As the advantages are demonstrated by the examples at hand there is a greater demand for such regulation. If the riparian question could be satisfactorily adjusted there would be, in the writer's opinion, a development along these lines comparable to that which has taken place in many of the other Western States. California has not of course built up an organization similar to that in Colorado, Wyoming, or Idaho. Its water master efforts thus far have been more or less sporadic and of small consequence compared to other items of the Division of Water Rights' work. Many cases come up as emergencies for short times only and so far the work has not been large enough to warrant a separate organization. The time is approaching, however, when this will have to be provided and it is gratifying to have available the experience and precedent of the other States that have studied these problems for so many years.

Herminghaus Decision

Returning to the most important question affecting California water rights, that of riparian rights, the State Supreme Court during the past thirty years had considerably narrowed this principle by limiting riparian areas by insisting, in some cases, on actual need of water. However, in the latest case, that of Herminghaus *vs.* Southern California Edison Company, the Court in a lengthy opinion made a sweeping declaration in favor of riparian owners, stating among other things that the use of the full flow of a river in order to afford the natural diversion of a small part of its waters through overflow was a reasonable riparian use. In the particular case in question this meant that the flood flow of the San Joaquin River, 20 000 sec-ft., could be required for the natural irrigation of the Herminghaus lands while it was admitted that 180 sec-ft. would be sufficient if artificial means of diversion were used. This principle would mean in effect that storage could not be made on a stream where a lower riparian owner demanded the full flow of the stream for the natural irrigation of his lands.

As on its face this decision appeared to make it possible for riparian owners to prevent up-stream storage, it has been widely discussed and solutions of various kinds have been advanced. For the past six years the State of California has been conducting investigations with a view to devising a plan for the most complete utilization of its water resources. The key-

note of such a plan is the storage of flood waters, as the natural flow of the streams of the State during the irrigation season is already practically taken up. This decision then became of importance to the State itself in this connection.

The 1927 Legislature actively took up the question and considered many proposed Constitutional amendments designed to clear the situation. After a hard battle in the Legislature one such amendment was passed and will be voted on at the next general election in November, 1928. It is known as Constitutional Amendment No. 7 and provides that water rights in California shall be limited to such water as shall be reasonably required for the beneficial use to be served and shall not extend to unreasonable use, unreasonable method of use, or unreasonable method of diversion. While this measure is aimed directly at the doctrine declared by the Herminghaus decision, it will be seen that its effect if passed will depend entirely upon the definition of "reasonable" as applied to methods of diversion and use of water. This can doubtless be clarified by subsequent legislation if the amendment is adopted.

The writer can well imagine that to water users in Colorado, Utah, Wyoming, and the other States which have so fully utilized their water resources, it is incomprehensible that the State of California should countenance such a wasteful legal policy as that set forth in the Herminghaus decision. It is now before the people of California and their decision is awaited.

PRESENT TENDENCIES IN WATER ADMINISTRATION

By George M. Bacon,* M. Am. Soc. C. E.

The discussion of present tendencies in water administration will be confined to the practice and experience of the State of Utah. It is believed that as much value will attach to a specific case as to a general treatment of the subject. By considering a concrete example, and comparing other experiences with it, lessons of value can be drawn as easily as by trying to deduce such lessons from a more general and broad treatment of the subject.

When the pioneers first settled in Utah and commenced to irrigate, the problem of distribution was one of mutual agreement. Later, as the waters of the various streams became fully appropriated, the inevitable litigation set in, resulting in the usual practice of having the Court that decided the rights administer them through a water commissioner of its own appointment. With the spread of irrigation throughout the State, and the creation of the office of State Engineer, it was recognized that many advantages would come through having the administration centered, if possible, under one single authority.

To effect this a statute was passed giving the State Engineer authority, in his discretion, to establish water districts and appoint water commissioners to distribute the waters. There followed, as might be expected, a conflict between the jurisdiction and authority of the Courts and that of the State Engineer in the administration of water rights. This controversy was decided by the Supreme Court of Utah in favor of the State Engineer, who was held to have administrative charge of the distribution of water in the State and who, therefore, could, where the proper formalities had been complied with, supersede the Court Commissioner by one of his own appointment.

Along with this gradual development there grew up the most important theory that the determination of the rights was solely a matter for the Courts, and that the administration of these rights was in the hands of the State Engineer. This division and separation of authority is thoroughly sound and of great practical value in avoiding the overlapping of functions which must always produce confusion.

One of the chief administrative difficulties in Utah arises from the fact that at present water rights are of two distinct origins. The older rights are the result of appropriation merely by beneficial use and, in a great many cases, are not of record anywhere. Where these rights have been the subject of litigation there followed a Court decree, but these Court decrees are very seldom definite, and some of the older ones are so faulty that they prevent any intelligent distribution of the water according to their terms.

In 1903 the law was passed requiring application to the State Engineer as an essential for the acquirement of right to unappropriated water in Utah. After the passage of this law an additional complication arose, owing to the

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fact that a few water users scattered over the State, either knowingly or ignorantly, ignored this law and attempted to acquire rights merely by the diversion of water to beneficial use. This situation existed for more than twenty years, until it was finally cleared up by a decision of the Supreme Court of Utah in 1925, which held that no water right initiated subsequent to 1903 was valid unless acquired through application to the office of the State Engineer.

It is obvious that a full and complete record of water rights initiated subsequent to 1903 exists in the office of the State Engineer; but as to the rights acquired prior to 1903 the record, if any, is scattered through the files of the various District Courts in the form of decrees. A beginning of remedying this unsatisfactory condition was made by the passage of a law in 1919 prescribing a method of adjudicating water rights through co-operation with the office of the State Engineer. In the earlier litigation there was always a great mass of conflicting testimony and no impartial study or investigation of any sort.

The new law is most valuable in that it provides for this impartial study and investigation by the State Engineer's office with a definite recommendation to the Court in the form of a proposed determination as to what, in the judgment of the State Engineer, the rights should be. Ample protection to the water users is secured by a provision that each must be served with a copy of this determination, and that any water user may object to the right proposed for him and have his day in Court to present such evidence as he may desire, showing the incorrectness of the right proposed.

Considerable progress in adjudicating rights has been made under the new law which has proved very satisfactory to the water users in its technical results, and also, in that its cost is but a fraction of the cost of litigation under the old method.

Real success of the administration of water rights is dependent wholly on an exact description and definition of such right. In the various districts where adjudications have taken place under the law of 1919 this information is available, being equally at the disposal of the water commissioner and of the various water users, with the result that the matter is wholly in the open and each user knows just what his neighbor's rights are. This makes for the essential understanding which is necessary for co-operation between all concerned and the old differences and frictions are practically eliminated.

In the older districts where the rights are less clearly defined there are complications not only of proper physical distribution but of the assessment of its cost as well. Continued efforts are necessary to educate the water users in such districts as to the difference between an actual right and the quantity of water available to supply such right.

With practically no exception all the water sources in Utah are heavily over-appropriated, which means that the problem of distribution is entirely different in a dry year from that in a wet year. In a dry year the later appropriations must have their supply cut out as the season of high water draws to an end, and this means that all rights are not of equal value.

Therefore, for the purpose of assessment, the rights are frequently divided into two or three classes and the levy per acre is made considerably less in the case of those rights that fail of supply at the end of the high-water season.

While the fundamental law of water rights, "first in time first in right", is in general controlling, there are a number of exceptions in a dry season. Besides this fundamental law is a secondary division according to use, into domestic, irrigation, power, and miscellaneous rights. The laws of Utah imply that the matter of priority does not necessarily run to the extent of withholding a junior domestic right to supply a senior irrigation right, or a junior irrigation right to supply a senior miscellaneous right. An added complication is found in many of the older Court decrees where the dates or priority were uncertain of record and the Court in its decree divided rights into classes. Where this situation exists it is also usual to deviate from the classes to take into consideration the secondary priority as already outlined.

It is possible that with future developments some method may be found of doing away with the classification in these old Court decrees, at least to the extent of having one fixed early date covering the unknown and uncertain priorities and, after this, definite dates for the other rights. At the time the Utah adjudication law of 1919 was passed there did not seem to be sufficient progressive sentiment to write into it a provision which reviewed in whole these old Court decrees and substituted for their provisions the modern and latest description of water rights. At present, in formulating the State Engineer's proposed determination, it is necessary to follow quite closely the terms of the Court decrees, except where these in actual practice have been found by the irrigators to be so unsatisfactory that they are glad to have them changed.

There is another extremely important phase of administration which can only be mentioned. This has to do with the administration of such water rights as are of record ownership in some agency of the Federal Government. Two types of this situation are the distribution of waters owned by the United States Indian Service or its successors in interest, and water rights and lands embraced in Federal Reclamation Projects. At present, there is not the slightest attempt on the part of the State to take over the Federal control of distribution any more than there is active effort by the State Engineer to supersede by his commissioners the various Court commissioners still distributing water on numerous streams in the State. On the other hand, no good can come, either of division of authority or indefinitness of such authority. In view of the accepted theory that States control water rights and of the tendency, observed from time to time, toward centralization under Federal control, a definite policy must be pursued by States in the arid region with reference to retaining these particular rights which have been regarded as definitely settled by decision of the United States Supreme Court in favor of the State. If attempts to view this matter as not finally settled are ignored, there is always the possibility that the question might be re-opened and under circumstances which, in the future, might cause the loss of valuable rights.

It is hoped that the foregoing brief review has indicated a plain tendency for administration of water rights to approach closely the ideal state expressed by the following controlling conditions

(*a*) Complete separation and definition of powers affecting water rights; one body, namely the Courts, to determine the rights; and the administration and distribution of these rights by another agency, the State Engineer.

(*b*) The abolition of all Court commissioners, their places to be taken by commissioners appointed by the State Engineer, after consultation with the water users.

(*c*) A method of assessments based strictly on water rights with such adjustment as will make them agree approximately with the value of the right in terms of available supply.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

RETURN WATER AND DRAINAGE RECOVERY FROM IRRIGATION*

A SYMPOSIUM

PAGE

Note.—Written discussion on this Symposium will be closed in August, 1928.

* Presented at the meeting of the Irrigation Division at Denver, Colo., July 14, 1927.

RETURN WATER, NORTH PLATTE RIVER, NEBRASKA

By R. H. Willis,* Esq.

Synopsis

The subject of return water is an interesting problem in the North Platte Basin. To determine, with a reasonable degree of accuracy, the quantity of return water in any irrigated area it is necessary that full co-operation be maintained between the State Department and the managers of those projects diverting water from the stream. Experience shows that it takes several years to obtain the necessary co-operation from project managers. In Nebraska, it has not yet been fully attained.

Quantities

The quantity of water returning to the North Platte River, after being diverted for irrigation, is amazing to those who have not observed the habits of this stream. The mean annual flow for the past thirty years, at North Platte, Nebr., is 2 294 000 acre-ft. This mean is not likely to change materially by having more years of record. Approximately, 1 200 000 acre-ft. have been diverted from the stream during each of the seasons of 1925, 1926, and 1927, between Whalen, Wyo., and Bridgeport, Nebr., and were conducted to the farming lands in the basin over which it was spread for the production of crops. About 65% of this water came back to the stream. This water is known as "return water", "return flow", "seepage", and "unused water."

In 1909, there were probably fewer than 50 000 acres under irrigation in the valley west of Bridgeport. Not until 1912 did the visible return flow become perceptible. After 1909 much additional land was put under irrigation between Whalen and Bridgeport. By 1919 the total had reached 233 000 acres. At present, (1927), the irrigated area in the valley comprises approximately 350 000 acres.

Elements Contributing to Return Flow

Since 1910 there have been two main factors affecting the habit of the river, namely, the storage of flood water in the Pathfinder Reservoir; and the increased diversions from the stream. The reservoir serves as a direct retard of the flow of the stream from the non-irrigation to the irrigation season. The effects which flood storage and return flow have on the lower section of the North Platte River are shown graphically in Fig. 1. The mean monthly flow before and after the completion of the Pathfinder Reservoir is shown for each month of the year. It will be noted that since the construction of the reservoir the mean flow for May and June has been approximately 50% less than the mean prior to that time. For July, the mean flow has been

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30% less; for August, 95% more; for September, about 225% more; and for October, 215% more than the mean flow prior to the beginning of storage in the Pathfinder Reservoir. This increased flow during the months cited is the result of the application of water to the irrigated area between Whalen and North Platte.

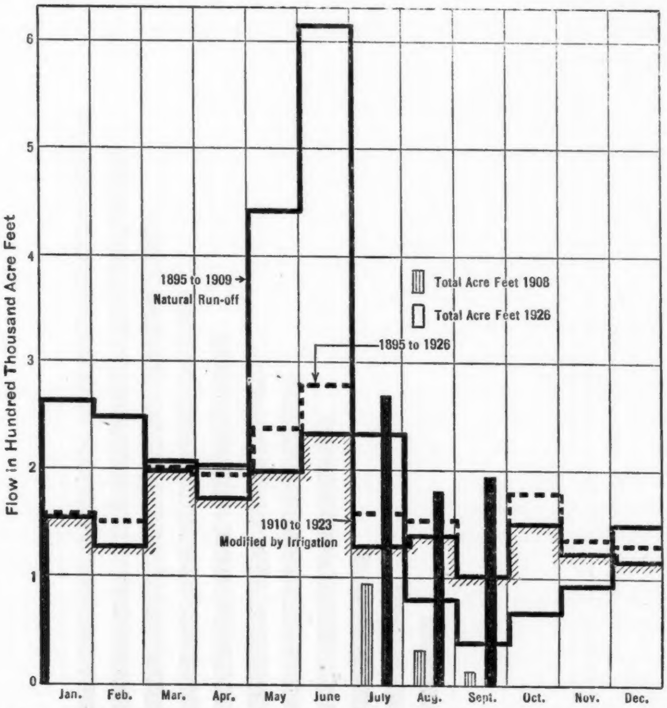

FIG. 1.—MEAN MONTHLY DISCHARGE OF NORTH PLATTE RIVER, AT NORTH PLATTE, NEBR.

The increased diversion of irrigation water has been the major contribution to the return flow. It is generally admitted that approximately 1 sec-ft. of water is consumed for every 3 sec-ft. diverted. The 2 sec-ft. become return flow after serving as a vehicle for carrying the 1 sec-ft. consumed. Part of the unused water is recovered by drainage canals. Although there are many miles of these canals in use in the North Platte Valley, many more miles must be constructed before return water is efficiently brought back to the stream. It must be kept in mind that some of the unused water returns to the stream through channels other than the drainage ditches. Not all the unused water is returned through drainage canals. Probably the major portion is returned through deep percolation designated as "invisible return flow."

RECORDS OF FLOW, RAINFALL, AND TEMPERATURE

Visible return flow, as herein used, includes some waste water. Nearly all projects use drainage canals occasionally to conduct excess water back

to the stream, and it has not been practicable to separate the waste water from the seepage. There is also surface water from irrigated fields bordering along the drainage canals that must be taken as visible return flow. The visible return flow of the North Platte River between the Wyoming-Nebraska Line and Bridgeport amounted to 630 000 acre-ft. for 1926. Fig. 2 shows the yearly quantity of visible return flow each year from 1912. The three rather pronounced jumps—1916, 1919, and 1924—are accounted for probably by preceding wet years.

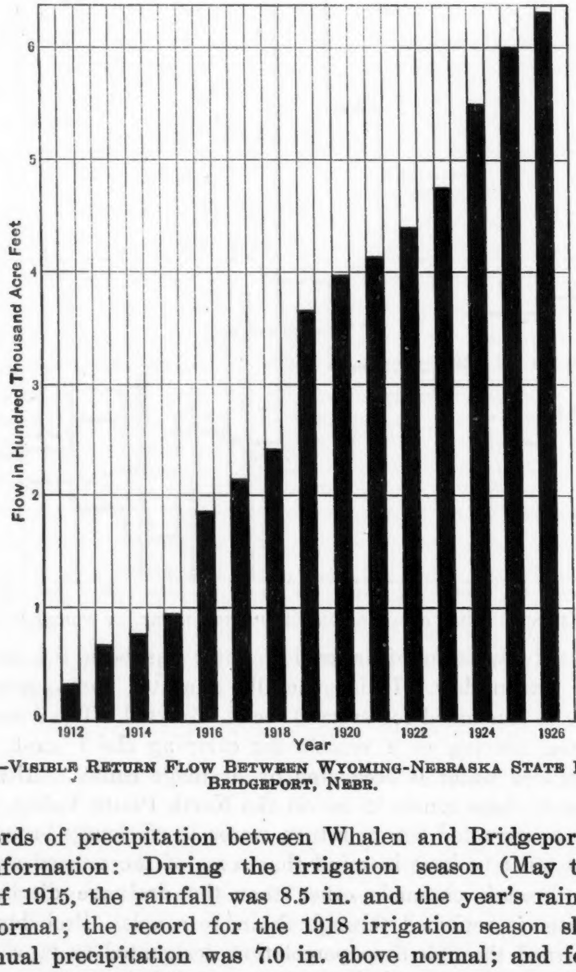

FIG. 2.—VISIBLE RETURN FLOW BETWEEN WYOMING-NEBRASKA STATE LINE AND BRIDGEPORT, NEBR.

The records of precipitation between Whalen and Bridgeport furnish the following information: During the irrigation season (May to September, inclusive) of 1915, the rainfall was 8.5 in. and the year's rainfall was 11.6 in. above normal; the record for the 1918 irrigation season showed 5.3 in. and the annual precipitation was 7.0 in. above normal; and for the season of 1923 the recorded rainfall was 4.6 in. and the annual quantity was 4.4 in. above normal. These are the greatest precipitations recorded since 1912, and the three wet years preceded the years that show the marked gain. The years 1916, 1919, and 1924 show the lowest precipitation since 1912, with the exception of 1914, which was 6.0 in. below normal.

The temperature during the irrigation seasons was below normal in 1916 and 1924, and considerably above normal in 1919. High temperature and hot winds diminish the visible flow. In Western Nebraska water is diverted for irrigation between May 1 and October 1. The diversions are greatest in July and August, and the return flow, in September and October, the lowest being about the last of April. The quantity of visible return flow available for use during the irrigation season of five months is approximately 48% of the annual visible return flow.

Variations in Return Flow

At Whalen, the United States Bureau of Reclamation operates and maintains a concrete diversion dam. Immediately above the dam two large canals, one on each side of the river, divert from the river during the irrigation seasons a quantity of water varying from 2 000 to 3 500 sec-ft. Special efforts were made to procure accurate data on the return flow between Whalen and Bridgeport during the years 1925, 1926, and 1927. There was a gradual increase in return flow from station to station down the river. For example, in August, 1925, it was found that there was no return flow at Whalen. The flow gradually increased at successive stations down the river until at Bridgeport, a distance of 93 miles, the mean daily return flow was 1 869 sec-ft., of which 57% was visible return flow. The remainder was invisible coming into the river from deep percolation and probably not all should be classed as return water. Rainfall in the basin contributes considerably to the invisible flow and causes it to vary. It is impossible to determine what part is rainfall.

The quantity of water returning in a comparatively short time is remarkable. Fig. 3 shows graphically the marked increase. This diagram includes both visible and invisible flow. There is a very great increase in 1926, and the records for 1927 confirm the 1926 curve. The precipitations for 1925, 1926, and 1927 were practically normal. The temperature was above normal in 1925 and 1926 and considerably below normal in 1927. The diversions for 1925, 1926, and 1927, between Whalen and Bridgeport, were 1 229 000, 1 935 000, and 1 171 000 acre-ft., respectively. The visible return flow shows in Fig. 2, a uniform increase for these three years. The astonishing increase for 1926, as shown in Fig. 3, is the invisible flow. The invisible flow comprises return water through deep percolation from the irrigated area and precipitation through deep percolation from the entire area of the drainage basin. Years of large precipitation and low temperatures are favorable to greater invisible flow.

No storage water was available for use in the valley until after the completion of the Pathfinder Reservoir in 1910. Since that year storage water has been used to supplement the direct flow. During the 1927 season, storage water was used on approximately 184 000 acres between July 1 and September 30. The direct flow was used on the remaining irrigated area between May 1 and September 30. The direct flow comprises the natural head-waters of the Platte Rivers, together with the return water from the irrigated area

of the basin. The return water has been sufficient for all diversions made east of Mitchell, Nebr., for the past three years or more. The return flow between Bridgeport and Overton, Nebr., comprises only a small part of the

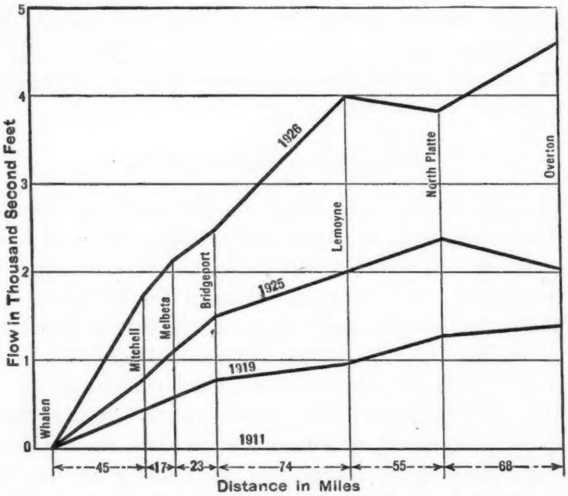

FIG. 3.—TOTAL ACCUMULATIVE RETURN FLOW, FIVE MONTHS.

total flow of the stream for the reason that the irrigated area between the two stations is comparatively small. During the seasons of 1925, 1926, and 1927, about 270 000 acre-ft. were diverted for irrigation purposes in the territory east of Bridgeport.

DRAINAGE RECOVERY FROM IRRIGATION

BY D. W. MURPHY,* M. AM. SOC. C. E.

SYNOPSIS

In the practice of irrigation a part of the water applied sinks below the zone of plant growth and is wasted. It is possible, in many instances, to recover a portion of this underground waste through drainage. The quantity that can be recovered depends largely on the character of the underground formation.

The feasibility of recovering irrigation wastes through drainage depends on the cost of draining the water out of the subsoil and its value at the point where it can be collected and discharged. On areas where drainage is necessary to protect soils from becoming water-logged and alkaline, the value, if any, of the water recovered is a net gain.

In the Salt River Valley in Arizona the water pumped to drain and protect the soils is a valuable asset: First, for supplementing the gravity supply during dry years; and, second, for sale for use on adjacent lands.

WHAT IS MEANT BY DRAINAGE RECOVERY

The term, "drainage recovery", like many others used in irrigation practice, is susceptible of various interpretations. In this paper it is applied to waters used in irrigation, which are not taken up by plants or evaporation and so can be removed from the soils through drainage and made available for further use. It is, so to speak, the underground waste from irrigation which may be recovered from the soils before it finds its way to some natural watercourse. Surface waste from irrigation, that is, waters which merely flow over the lands and off them as surface flow, are not included.

Return flow from irrigation differs materially from drainage recovery. It is water that finds its way through the soils downward and by lateral percolation to natural watercourses. On the other hand drainage recovery may, and ordinarily does, reduce return flow through cutting off all or a part of the water that would otherwise find its way to natural watercourses. It tends also to prevent evaporation from the surface by maintaining the water-table at a lower elevation and thus reducing the quantity of water brought to the surface through capillary action.

CONSUMPTIVE USE OF WATER

The total quantity of water applied in irrigation may be stated in terms of consumptive use by plants, of surface evaporation, and of waste, both surface and underground. Drainage recovery is dependent, it may be said, primarily

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on the last factor, underground waste. The amount of underground waste varies greatly for different conditions. The adequacy of a water supply, its cost, the method of application, and the character of soils all may influence it.

From the few determinations that have been made on the consumptive use by plants, it appears that a large part of the water applied in irrigation is lost either through evaporation or waste. In studies made over large areas in Southern California, the conclusion was reached that 1.82 acre-ft. per acre per year represented the average consumptive use by plants. From this, it appears that with ordinary irrigation a large part of the water applied is lost or wasted, so far as the area to which it is applied is concerned. It indicates also the probability of considerable underground waste and the possibility of large quantities being recovered through drainage.

Water That May Be Recovered

The quantity of irrigation water that can be recovered can never exceed that of underground waste, and it may be but a small, even a negligible, part of such waste. The factors that control the amount of recovery, assuming that the underground waste is a material quantity, generally include soils and the character of the underground formation. It is possible to conceive of an area lying in a closed rock basin out of which no waste from irrigation can escape. In such a case it is possible, through drainage, by pumps and wells, to recover all, or practically all, the irrigation waste from the given area.

The same is also true, but in a somewhat lesser degree, in many valleys with relatively narrow outlets. Practically all the waste into such valleys can be pumped out by a proper and judicious placing of drainage wells. Such areas exist at many places in the Valley of the Rio Grande, and also in those of other Western rivers. The conditions necessary for this sort of action are first a closed or nearly closed valley for holding the irrigation water; and, second, a valley fill sufficiently pervious to permit relatively free passage of water through it and to serve as a medium in which the waste waters may be collected. These represent extreme conditions favorable for drainage recovery.

It is equally possible also to conceive of an area not bounded by any rock or other impervious materials, from which underground waters can escape with relative freedom. Where this is the case the irrigation waste flows away more freely, and little or none of it can be collected by drainage works. Such areas are sometimes found on sandy or gravelly mesas adjacent to streams, and in valleys which widen rather than contract toward their lower ends.

Another condition which makes difficult the collection of drainage water is a shallow, comparatively pervious top stratum underlaid by impervious materials. The drainage of such a formation ordinarily must be accomplished by cutting into the impervious materials and collecting the waste water in such excavations by the slow process of flow or percolation over the top of the impervious stratum. Such a condition permits of little storage capacity in the soils. Furthermore, it is difficult to collect the drainage water in large quantities, except through a long system of drains. In such cases the drainage discharge may be at the extreme lower portion of the irrigable area.

Feasibility of Recovery by Drainage

There are several practical and economic questions to be considered in connection with drainage recovery of irrigation water. Among these may be mentioned the following:

(1) The quantity of water that can be recovered.
(2) The quality of water, that is, its fitness for use in irrigation.
(3) The cost of recovery.
(4) The location at which water can be delivered.

The quantity of water that can be recovered may vary from nothing to possibly as high as 20 or 25% of that applied in irrigation. It may even exceed this percentage on limited areas. Large underground wastes in irrigation are most commonly experienced during the early stages of a development. This is due in large part to the limited experience of settlers in irrigation and also to the more abundant water supply available in the first stages for development. Experience together with the greater demand and consequent increased value of water, tends gradually to reduce waste in irrigation. For this reason a water supply which depends wholly, or in large part, on drainage recovery may be reduced in future years, due to more economic and better use of water in irrigation.

It is clear that irrigation waste is due, in large measure, to man's inability to deliver water to land in the most efficient manner. Further, it is impossible to predict with any degree of certainty how much water can be recovered from a given area in advance of actual irrigation and drainage of that area. Information concerning the quantity recovered from one area is of little value for another, due to the many variable factors in each.

Quality of Drainage Water

The character of drainage water must be given consideration in every case where it is to be used for irrigation purposes. This feature is difficult, due to the fact that there is no fixed standard by which the quality can be readily compared. There was a time when soil chemists were willing to state with some degree of positiveness that water containing more than a certain amount of harmful alkali salts was unfit for use in irrigation. Many contradictory experiences have somewhat modified these ideas, so that now most soil chemists and engineers are willing to admit that water containing slightly more than this specified amount may not be unfit for use and that if it contains less than the specified amount, it may not be good.

Cost of Drainage and Value of Water Recovered

The cost of draining irrigable areas ordinarily is an obligation that must be met for the protection of lands, whether or not the water drained off can be utilized. Up to a certain point the cost of drainage works cannot be charged against the supply attained through drainage. It frequently, happens however, that the expense of conveying a drainage supply to a locality where it can be utilized is high, and consideration must be given to it. This leads at once to the fourth consideration as to where the recovered drainage water can be delivered.

In many areas of extreme aridity where the water supply is the controlling factor in determining the extent of agricultural and other development, attention should, and is being, given to drainage recovery to supplement other supplies that are well nigh exhausted. In such cases the value of water is generally sufficient to justify any reasonable expense that must be incurred in order to obtain a supply.

In areas where there is more water than is required to irrigate the available supply of land, economic questions are of first importance in drainage recovery. In any event each individual case must be given independent study and consideration. There is not sufficient experience on the subject to permit of such generalities as that all drainage water should be utilized. Neither can it be said that it is not feasible to recover drainage water, since it has been shown through actual experience that in some localities it is economically and otherwise feasible.

It is evident, in the truly arid regions, that agricultural and other developments will require all the available water supply within a few years. It is necessary in such areas to give consideration to conserving the entire supply. This will require a curtailment of irrigation water or a recovery of such wastes through some form of drainage. It is reasonable in this connection to assume that no single method can or will be used in such conservation, but that waste from irrigation will be reduced to a minimum through better methods of use, and that unavoidable waste will be recovered as far as practicable.

Example of Drainage Recovery

The most noted instance of recovery of water through drainage that has come within the writer's experience is in the Salt River Valley in Arizona. The irrigable area within the project is about 200 000 acres. After irrigation had been in progress for a number of years the water-table rose, over a large part of the area, sufficient to threaten the irrigability of the soils; and over a considerable area, sufficient to render them unfit for cultivation, due to water-logging and the accumulation of harmful alkali salts on or near the surface.

The plan considered most feasible for draining the valley was by means of wells sunk to depths varying generally from 150 to 300 ft. The formation of the valley is such that pumping from deep wells has a direct effect on the elevation of the water-table; that is, the water-bearing materials of the upper and lower portions of the valley fill are sufficiently connected to cause a general lowering of the ground-water by pumping from deep wells.

In deciding on the plan of using wells for drainage, it was the intention that the water pumped from the soils would, in so far as possible, be used to augment the natural irrigation supply for the project which comes from Roosevelt Reservoir. With this end in view, wells were located, consistent with drainage requirements, so that water could be pumped directly into canals or laterals and used on the lands of the project.

On a part of the area where the pumped waters have not been regarded as needed for supplementing the gravity supply, they have been sold to be used

on outside lands. The following tabulation gives the amounts pumped by years, from 1919 to 1926:

Year.	Acre-feet.
1919–20	55 833
1920–21	55 368
1921–22	109 211
1922–23	152 464
1923–24	167 000
1924–25	309 703
1925–26	251 432

All or practically all this pumped water was used for irrigation to supplement the gravity supply.

The system of drainage by wells has an advantage over any system of drains for water recovery. A heavy draft can be made on the ground-water supply during the irrigation season or during years of short gravity supply, and pumping can be discontinued during the non-irrigation season or when pumped water is not needed. By this method of operation the water-table can be maintained at all times at a safe distance below the surface and, at the same time, the underground storage supply can be largely conserved.

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PAPERS AND DISCUSSIONS

TRANS-MOUNTAIN DIVERSIONS*

A SYMPOSIUM

Note.—Written discussion on this Symposium will be closed in **August, 1928.**

* Presented at the meeting of the Irrigation Division at Denver, Colo., July 14, 1927.

INTER-WATER-SHED DIVERSIONS BY TUNNELS ON UNITED STATES RECLAMATION PROJECTS

By E. B. Debler*, M. Am. Soc. C. E.

Synopsis

Trans-mountain diversions, from the Colorado River drainage area to adjacent basins for irrigation and other uses, have received much consideration in connection with the proposed construction of the Boulder Canyon Reservoir and the proposed compact for the allocation of the Colorado River waters. The aggregate total of such diversions is expected to exceed, in the ultimate, 500 000 acre-ft. per year. One of the most important of these is the Strawberry Tunnel Project, constructed by the U. S. Bureau of Reclamation to irrigate lands in the vicinity of Provo, Utah. Another trans-mountain diversion, although not diverting away from the Colorado River Basin, is the Uncompahgre Tunnel, between Uncompahgre and Gunnison Rivers in Colorado, serving the Uncompahgre Project. This 6-mile tunnel, at the time of its construction (beginning in 1904) was the longest tunnel in the United States, and the most imposing undertaking up to that time in connection with irrigation.

The Gunnison Tunnel

History.—The Uncompahgre River is a tributary of the Gunnison River which, in turn, enters the Colorado River at Grand Junction, Colo. The lower portion of the Uncompahgre traverses a broad fertile valley without sufficient rainfall for crop production and with local stream flows inadequate for the irrigation of the valley. Paralleling this valley and separated from it by a mountainous ridge lies the Gunnison River (Fig. 1) with a flow very much greater than that of the Uncompahgre and devoid of bordering irrigable areas capable of using material portions of its flow. Furthermore, the Gunnison River lies at uniformly higher levels than the Uncompahgre Valley.

Rapid settlement of the valley started in 1884, and numerous large irrigation canals were constructed before stream discharge data had become available. It was soon found that with 100 000 acres under irrigation less than 30 000 acres could reasonably be expected to receive an adequate water supply from local sources.

The diversion of Gunnison River water to the valley was conceived in 1890 by Mr. L. C. Gauzon. In 1894, surveys were undertaken by popular subscription. In 1901, the Colorado State Legislature appropriated $25 000 for the location and construction of a Gunnison tunnel but, in 1902, these funds were exhausted after a tunnel had been driven 900 ft. In 1901,

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investigation of the Gunnison Tunnel plan was undertaken by the U. S. Geological Survey under the direction of A. L. Fellows, M. Am. Soc. C. E., the work in 1902 being transferred to the newly created Reclamation Service. A new tunnel location was adopted in 1904, following extensive surveys.

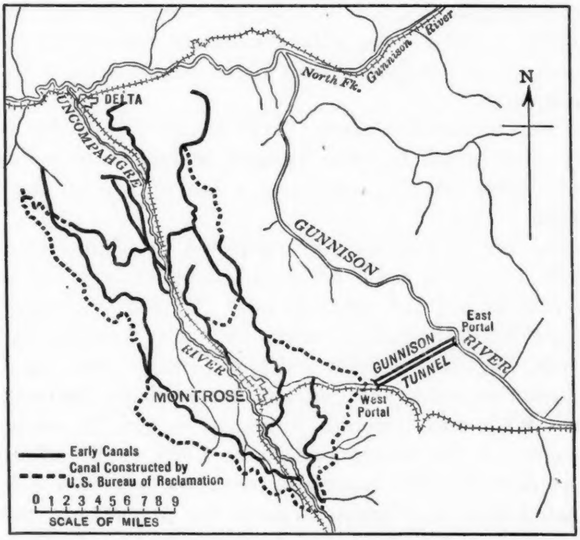

FIG. 1.—MAP OF GUNNISON TUNNEL AND UNCOMPAGHRE PROJECT, COLORADO.

Plans and specifications for the tunnel were completed in July, 1904, and in the October following, a contract was awarded the Taylor-Moore Construction Company, of Hillsboro, Tex. The contractor withdrew from the work in the following May, after completing about 4.3% of it. In August, 1905, an advertisement was issued for bids on the completion of the work, but those received were considered excessive. The work was then undertaken by the Reclamation Service with plant and equipment turned over by the defaulting contractor, with subsequent additions thereto. The tunnel was holed through on July 6, 1909, and enlarged to full section in February, 1910.

Tunnel Section and Materials.—The tunnel has a length between portals of 5.8 miles. In general, the interior section presents vertical side-walls, 11 ft. apart, surmounted by an arch of 6 to 8 ft. radius, with the crown 12 ft. above the floor. The principal variations during construction related to changes in the radius of the arch to care for changing conditions.

The materials encountered, starting from the west portal, were as follows:

For 2 000 ft., alluvial deposits of sand, gravel, and clay, all water-bearing and very heavy.

For the next 1200 ft., the same materials with a gradually rising bed of hard shale requiring careful timbering to support the roof and blasting to remove the bench.

The following 10 000 ft., black shale containing inflammable and explosive gases, but dry.

Then 2000 ft. of badly faulted and shattered mixtures of sedimentary rocks and coal containing gases, and much hot and cold water.

Finally 15 380 ft. of metamorphic granite, gneiss, and schist of greatly varying hardness with frequent seams carrying much water.

Construction Plan.—The general plan of construction in the alluvial formations was by means of a 4 by 6-ft. top entry, followed by the opening of the arch haunches and sides, with subsequent removal of the bench. In hard formations a top heading, 6 to 8 ft. in height and of full width, was followed by benching.

Timbering was required on about 18 000 ft., or 60% of the tunnel length, almost evenly distributed between ground materials requiring immediate timbering and harder shales permitting a lag of several days behind the bench excavation.

The concrete lining was largely completed by June, 1910, with some lining placed in 1912, and small amounts subsequently. The original plans contemplated full lining, but actually only the floor is wholly lined, with the sides and top lined throughout somewhat more than one-half the length. The tunnel costs, including portals and portal cuts, were roughly $2 900 000, including construction roads and engineering, or about $500 000 per mile.

Utilization.—With full concrete lining as originally contemplated, the estimated capacity of the tunnel was to be 1 300 sec-ft. With the partial lining thus far installed, the maximum diversion has been 975 sec-ft. The original estimated area of irrigable lands in the project was, roughly 140 000 acres. Modifications of the planned distribution system, following the acquisition of numerous existing canals and their adaptation to the project in preference to the construction of new canals, together with seepage developments and the inferior quality of some of the lands originally included, leaves an area of good irrigable lands amounting to 95 000 acres, of which 65 000 acres are now irrigated.

The principal impediment to extension of the irrigated area is widespread seepage without sufficient general interest by the land owners for the adoption of a comprehensive drainage program. For the irrigated area, the delivery of water to the land has averaged 5.7 acre-ft. per acre annually. With a moderate improvement in the duty of water, the combined water supply available from the Uncompahgre River and through the Gunnison Tunnel in its present condition, is estimated to be sufficient for the irrigation of about 85 000 acres.

An increase in the tunnel capacity would not be reflected in a corresponding increase in water supply, as the present capacity is in many years greater than the flow of Gunnison River for varying periods in late summer. The Gunnison has no storage control, but an excellent reservoir site on Taylor Fork has been withdrawn from entry and set aside for use by the Uncompahgre Project in case it should become desirable to increase the low-water flow of the stream in connection with an increase in the tunnel capacity by the completion of the lining.

The annual diversions through the Gunnison Tunnel, all used for irrigation, have averaged 237 000 acre-ft. for the five-year period ending with

1926. The construction charge payable to the United States for the Gunnison Tunnel and for the construction and reconstruction of canals and laterals is $52 per acre. The principal crops are alfalfa, sugar-beets, and potatoes, with an average annual crop production at the farm of $44 per acre over the past ten years.

The Gunnison Tunnel is not properly a trans-mountain diversion as the term is usually understood, in that the irrigated area lies along a tributary of the parent stream from which the primary water supply is derived. It is, however, to date (1927), the longest tunnel built for irrigation purposes.

Strawberry Tunnel

History.—The Strawberry Valley Project (Fig. 2) comprises Strawberry Reservoir with a capacity of 255 000 acre-ft. located on Strawberry River, thirty miles easterly from Provo, Utah; the Strawberry Tunnel diverting reservoir waters westerly through the Colorado River-Great Basin Divide into the Spanish Fork water-shed; and a distribution system for lands in the vicinity of Springville, Spanish Fork, and Payson, Utah.

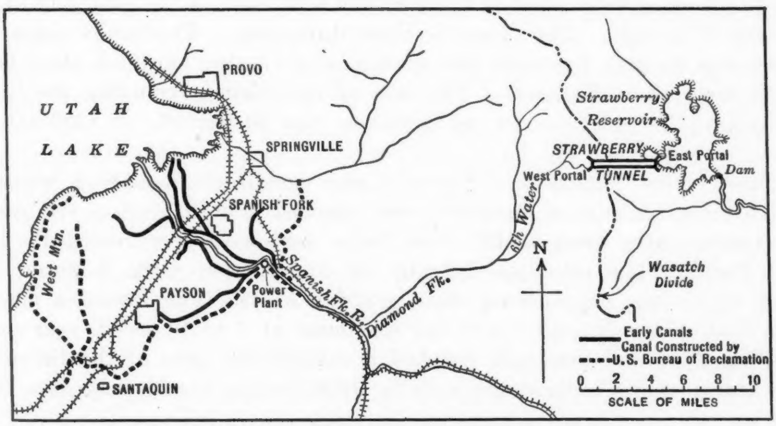

FIG. 2.—STRAWBERRY TUNNEL, STRAWBERRY PROJECT, UTAH.

The diversion of Strawberry River waters to the Spanish Fork Basin received active consideration by the State Engineer of Utah and by irrigators in the Basin about 1900. In 1902, a full survey of the plan was made by a company interested in irrigation storage and power development, but was rejected as too costly.

In 1903, an investigation of the plan was undertaken by the U. S. Reclamation Service at the request of interested citizens, but a number of factors slowed up progress. Among these were the location of the Strawberry watershed within the Uintah Indian Reservation; the extremely low run-off for a few years immediately preceding, indicating a comparatively high construction cost for the estimated yield of water; and lack of adequate interest by land owners to be benefited.

With the definite abandonment of plans to use Strawberry River waters within the Duchesne water-shed, the opening of the Uintah Indian Reservation, and increasing interest in irrigation, a petition for further consideration of the project was received in January, 1905, from about 1 200 citizens. Following the completion of arrangements for repayment of construction expenditures, commencement of work was authorized in March 6, 1906, and \$1 250 000 was set aside for this purpose from the Reclamation Fund. Work was immediately started on roads and telephone lines to the tunnel portals.

In August, 1906, bids were invited for the construction of the tunnel, but as no proposals were received, construction by force account was authorized in September, 1906, and promptly commenced at the West Portal. From July, 1907, to November, 1908, tunnel work was suspended awaiting the construction of a hydro-electric power plant on the lower reaches of Spanish Fork River to furnish power for tunnel construction. Thereafter it was resumed at the West Portal. In October, 1911, work was commenced at the East Portal. The tunnel was holed through on June 20, 1912, and the concrete lining was completed in November, 1912.

Tunnel Data.—The tunnel has a length of 3.6 miles, with an interior section of a flat floor, vertical sides, 7½ ft. apart and 6 ft. high, to the springing line of an arch 2 ft. high. The tunnel is lined throughout. Practically the entire length was through limestone and sandstone of varying hardness, shale being found for a short distance. The cost of the tunnel, including the lining and appropriate charges for hydro-power, was \$1 150 000, or \$320 000 per mile.

Construction Difficulties.—Severe winter conditions hampered construction to some extent, as snowfall often reached 200 in. during the season and temperatures down to 49° Fahr. below zero have been recorded at the East Portal. The principal difficulty in construction work, however, was water which was encountered about midway of the tunnel with a flow of more than 10 sec-ft. and which has continued at 7 to 10 sec-ft. ever since. Swelling shale and limestone wrecked a considerable part of the lining, so that heavy repairs became necessary by 1919, but no trouble has since been experienced.

Utilization—Strawberry Reservoir, with a capacity of 255 000 acre-ft. available for diversion through the tunnel, is formed by an earth-fill dam with a maximum height of 72 ft. and a length of 488 ft., across Strawberry River, and an auxiliary dam, 37 ft. high and 1 300 ft. long, across a low pass into the Indian Creek water-shed. Early investigations indicated that the cost of collecting water from streams on the north into the reservoir would be excessive in relation to the obtainable water. Indian Creek has been intercepted by a short canal, and an extension thereof to intercept Trail Hollow and other small streams can be made at moderate cost when sufficient demand develops for the water.

The safe annual yield of water from the Strawberry Reservoir is estimated at 80 000 acre-ft., with a possible increase to about 95 000 acre-ft. after complete development. The average draft on the reservoir from 1921 to 1926, inclusive, has been 84 400 acre-ft., which, with 6 000 acre-ft. per year from

tunnel leakage, has been added to the flow of Spanish Fork River, averaging 114 000 acre-ft. annually, for diversion by canals serving about 60 000 acres of land at the southeast border of Utah Lake.

The first canals from Spanish Fork River were constructed in 1851 by Mormon settlers. Long before 1900 serious shortages of water were common, with occasionally disastrous results from over-development of the natural flow of the stream. Large storage reservoir sites were not available on the water-shed, and storage of surplus waters on the stream would have infringed on irrigation rights in Utah Lake. The delivery of foreign waters thus solved a problem that might otherwise have resulted in endless litigation.

Water from the Strawberry Reservoir has been sold to the old canals at an average rate of about $45 per acre-ft. of annual use delivered in the stream. It was purchased for an average use of 1 acre-ft. of water for each acre to be irrigated, such water being supplemental to the natural flow. It has been customary in connection with the old canals to use such storage water on about one-half the farm area, the remainder of the area receiving a partial supply only and being devoted to appropriate crops.

The Federal Government constructed two new canals, the High Line Canal for the irrigation of about 20 000 acres of new lands lying above the older lands southerly from Spanish Fork River, and the Mapleton Lateral, primarily to deliver waters to Hobble Creek as a supplemental supply for the canals diverting therefrom. Under the High Line Canal, the average diversion is about 2½ acre-ft. per acre annually. This very high duty for the inter-mountain territory is influenced by the high cost of the water, construction charges on this project being $60 per acre.

Approximately, 10 000 acre-ft. of water from Strawberry Reservoir remain unsold at this time. The present price is $60 per acre-ft. delivered in the stream, with the demand rather light, as the principal areas which could profitably use the water also require drainage and the combined costs deter development.

The principal crops on the older lands of this project are canning vegetables, sugar-beets, and fruits. The newer lands are more largely devoted to alfalfa and grain. The average annual crop value for the project is $45 per acre.

The Strawberry Tunnel diversion is the largest single diversion out of the Colorado River water-shed and, in fact, constitutes roughly 70% of all such diversions now being made.

THE PROPOSED LOS ANGELES-COLORADO RIVER AQUEDUCT

BY H. A. VAN NORMAN,* M. AM. SOC. C. E.

SYNOPSIS

This paper discusses the need, in Southern California cities, for water from the Colorado River, which is the only large unappropriated supply near this district.

A description is given of several of the proposed routes that would ultimately divert 1 500 cu. ft. per sec. Two of these routes are unique in being gigantic pumping schemes, either of which would utilize a large block of power from the proposed Boulder Dam. Two others—gravity routes—are unusual in that they entail the driving of tunnels of heretofore undreamed of lengths.

Surveys have been extended into 18 000 sq. miles of territory in the Colorado desert previously unmapped.

Various types of aqueduct construction are discussed, as well as the economic problems related thereto.

A NEEDED IMPROVEMENT

The recently proposed Los Angeles-Colorado River Aqueduct to provide a municipal water supply from the Colorado River for the City of Los Angeles, and such other cities of Southern California as care to join with her in this enterprise, is unique in two respects. It is to be of greater carrying capacity than any previously constructed aqueduct, and it is not to be in its entirety of the usual gravity type.

The City of Los Angeles now has a population of 1 230 000, and has been growing at a rate which has averaged more than 70 000 inhabitants per year for the last 10 years. A careful investigation of its present water resources shows that it will reach the limit of its supply within 10 years.

Also, in the vicinity of Los Angeles are many suburban cities, some of which are increasing in population even faster, relatively, than that city itself. Some of these cities even now feel the pinch of water shortage, and many of them see the limit of their supply within a comparatively short time.

In order that these smaller cities may participate in the Los Angeles-Colorado River Project, an Act providing for the formation of Metropolitan Water Districts for the purpose of developing, storing, and distributing water for domestic purposes, was recently passed by the Legislature of the State. Under this Act such a district may be formed of the territory included within the corporate boundaries of any two or more municipalities.

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THE COLORADO RIVER SUPPLY

Unlike most other cities of its size, Los Angeles is at a great distance from any large source of water supply. Realizing this fact, and knowing the time it would take to construct an aqueduct from any distant source, William Mulholland, M. Am. Soc. C. E., Chief Engineer and General Manager of the Water Department, whose duties require him to look into problems of this nature, began, in 1922, to study the sources from which an additional supply might be obtained.

The Colorado River is practically the largest dependable supply situated at a distance that is not prohibitive in point of cost, and available with the fewest legal and engineering difficulties. Its distance from Los Angeles is similar to that of the Owens River, from which the city completed its first aqueduct about fifteen years ago.

The unappropriated surplus flow of the Colorado River is more than sufficient to meet the domestic needs of all the cities of Southern California. Unfortunately, this river in its lower reaches is situated at about the same general elevation as most of the territory to be served, and intervening between it and the Pacific Coast, lies a high mountain range, and a great area of relatively high desert. It is impractical to tunnel either in its entirety, and consequently, if the cost is to be kept within reasonable limits, the proposed aqueduct must be constructed at a level higher than its source. The problems to be solved are, therefore, mainly those of engineering.

With these general facts in mind, a party of engineers, headed by Mr. Mulholland, left Los Angeles on October 29, 1923, for a reconnaissance of the territory as far as the Colorado River, in order to investigate the feasibility of the project. Information obtained on this trip showed not only that an aqueduct could be constructed from the river to Los Angeles, but also that several routes were available.

Much of the country traversed consisted of vast desert waste land, separated by rugged mountain ranges acting as barriers. The greater part of the desert section was unmapped, and it soon became apparent that surveys would have to be made on an extensive scale. On the return of the party, machinery was set in motion for perfecting an organization, and making the necessary preliminary surveys.

SURVEYS

The geodetic control was based on the Texas-California arc of primary triangulation, of the U. S. Coast and Geodetic Survey. Such points of the Geological Survey as were available in the region, were used, and a system of triangulation control was adopted to extend the work into the new territory. North American datum for position and mean sea level for elevation were adopted as standards for this survey.

The grid system used by the U. S. Coast and Geodetic Survey for the polyconic projection of maps, was taken as a basis for platting. The One Hundred and Sixteenth Meridian, lying about midway between the Colorado River and the Pacific Coast, was taken as the meridian of reference for map work.

All the topographic mapping has been done by the plane-table method. With experienced topographers, many being former Geological Survey men, more than 18 000 sq. miles of territory have been mapped on scales of 5 000 and 10 000 ft. to the inch, and almost 1 000 sq. miles on the much larger scale of 1 000 ft. to the inch. A contour interval of 100 ft. was adopted for the small scale maps, and intervals of 20, 10, and 5 ft. for those of larger scale.

As a result of these extended surveys which are now nearing completion, the Department of Water and Power of the City of Los Angeles has recently constructed a large relief map of all this territory. On it may be delineated practically every route worthy of consideration for the proposed aqueduct.

Blythe Route

A route originating 15 or 20 miles above Blythe, Calif., is the one favored by Mr. Mulholland. This route has many attractive features, and most of the construction involved can be done with little difficulty. It has been surveyed and examined in greater detail than the others.

Commencing with an intake approximately due east from Los Angeles, and at practically the nearest point on the river to the city, it skirts the southern end of the Maria Mountains in Riverside County, California, for a distance of from 10 to 15 miles. Then, by successive lifts and grade conduits, alternately, the line is to be carried to the divide between the Colorado River Basin and the Coachella Valley of California. This divide is locally known as Shaver's Summit.

The total difference in elevation to be overcome in reaching Shaver's Summit is about 1 400 ft., to which must be added friction head. This would be accomplished in the first 75 miles of aqueduct. In order to overcome this difference in head with the least construction and operating difficulties, the lifting will be done in five stages. From Shaver's Summit to the end of the route, no further pumping will be required.

Between Shaver's Summit and Los Angeles, it will be necessary to construct approximately 35 miles of concrete-lined tunnels along the southwesterly face of the Little San Bernardino Mountains. There will also be a long tunnel under San Gorgonio Pass, varying from 13 to 27 miles in length, depending on its exact location, as yet not fully determined. From its westerly portal, a grade conduit and tunnels will complete the line to Los Angeles.

Along the Colorado River and parallel with the easterly part of this route are extensive gravel deposits, from which, it is believed, a sufficient quantity of clear water for the first few years of operation of the aqueduct can be obtained, either by constructing a large infiltration channel below the water-plane, or by pumping from wells suitably placed, or by both.

The adequacy of these gravels as a preliminary source of supply is to be thoroughly tested by pumping from a 2-mile section of full-sized infiltration canal, and from wells drilled at various points along the river front. Already about three-fourths of the length of the canal has been excavated, and twelve wells have been drilled to depths varying from 60 to 200 ft.

As a result of the construction of a high dam either at Black or Boulder Canyons, it is believed that when a greater supply is needed, the river will be sufficiently desilted at the proposed point of intake to permit of its being pumped directly into the aqueduct. If, however, this is not the case, it can readily be made clear and potable by the usual methods of mechanical filtration.

Black Canyon Route

A route originating at the Black Canyon Dam site has been surveyed in some detail. It is about 85 miles longer than the Blythe route, and involves greater construction difficulties. It also requires that the proposed Boulder Canyon Dam be constructed at the alternative Black Canyon site. As yet, the final location for this dam has not been definitely determined.

All lifts along this route would be near Black Canyon, and the corresponding total head would be greater than that required on the Blythe route. However, considerable return power could be generated at the westerly end of the aqueduct, making the net power consumed in pumping somewhat less than on the preceding route.

One inverted siphon, 13 miles in length, would be required. If this were constructed as a steel pressure line at the ground surface, it would be under a maximum head of 660 ft. If constructed as a tunnel with shafts, the head would, of course, be materially greater. The advisability of a tunnel siphon is questionable, because of the volcanic character of the rock formation. The cost of this route would be materially higher than that of the Blythe route.

Routes for Gravity Supply

In addition to the routes just described, two gravity routes have been suggested by other engineers. One has its point of intake about 110 miles up stream from the Boulder Canyon Dam site, and the other leaves the Colorado River a few miles below its junction with the San Juan, in Utah.

The former requires a dam at Bridge Canyon about 825 ft. in height above its foundation, in order to divert water into its aqueduct. Such a dam would create a reservoir storage of 6 200 000 acre-ft., and this reservoir would have to remain full at all times. The route would be 370 miles in length, including 170 miles of tunnels with one tunnel of 71 miles, and another of 80 miles. An inverted siphon across the Colorado River near Topcock, Calif., having a length of 12 miles and a maximum head of 1 300 ft. would also be necessary. This siphon would have to be constructed as a multi-barrel structure, in order to reduce the thickness of steel plate. If this route were adopted, an additional dam at the Glenn Canyon Dam site near Lee's Ferry, Ariz., would be required before many years had elapsed, to be utilized in postponing the time when the Bridge Canyon Reservoir would become filled with silt.

The other suggested gravity route would be more than 750 miles in length, and contain an even greater mileage of tunnels than the one described. It would remain on the northerly and westerly side of the Colorado River throughout its entire length. It is also proposed that this route be used for service other than domestic water supply in the States of Utah, Nevada, and California.

Sufficient investigation has been made of both of these gravity routes to show not only that they would be extremely expensive to construct, but that the time element would delay the delivery of water to the Coast cities of Southern California so long as to make it impracticable as a municipal or metropolitan water project. Both routes contain power recovery features, which would offset, in part, their great cost. At the Bridge Canyon Dam site, in particular, field investigations and measurements create a doubt as to the advisability of constructing a dam to a greater height than the top of the exposed igneous rock, or 630 ft. above the low water stage of the river. This would reduce the capacity of the reservoir to 3 100 000 acre-ft., and thereby greatly shorten its useful life as a de-silting medium.

Economies and Costs

All possible modifications of location, gradient, or lift, along the non-gravity routes, either have been or now are, under survey, and these will be given careful consideration in due course. Before the final determination of the particular route to be constructed is made, the economics of the situation require that much thought be given to many complex elements. Among them may be mentioned the following: The length of construction period, in years; the estimated cost of the proposed route completed; the cost as possibly modified by the omission of certain structures, such as units of pumping plants and force mains, which do not have to be constructed to full capacity at the start; the time factor introduced in the construction of the longest or most difficult tunnel, and the number and depths of shafts required; the number of lifts to be installed, and the lengths, sizes, and number of force mains comprising these lifts; the effective head available for return power; the cost of installing pumping plants, power plants, force mains, and pen-stocks; and the annual charges and pumping costs to be met in operating the routes under study.

The estimated cost of any of the several routes will vary with the hydraulic gradient selected. Those with flatter slopes, and correspondingly reduced velocity of water with increased cross-section, obviously will cost more to build. The cost of operating the pumping plants, on the other hand, will decrease with a reduction in the hydraulic gradient; consequently, these latter costs must be balanced in each case against interest charges on the proposed construction cost.

In operating an aqueduct of this character, the main item of cost is that of the power necessary for pumping. To date, this cost has not been fixed, although for the purpose of carrying on economic studies, it has been assumed that power will be available at the rate of 3 mills per kw.-hr. This is an estimated price for which power can be sold at the switchboard of a plant situated either at the Boulder or Black Canyon Dam sites, with a dam constructed 600 ft. in height above the low-water level of the river. In all studies due credit is given to an estimated possible income derived from the sale of return power at the westerly end of the proposed aqueduct, wherever head is available for that purpose.

As to the quantity of water required on the completion of the aqueduct project, studies have been made regarding the rate of growth of the population of all such cities of Southern California as may reasonably be expected to participate in the benefits. Their domestic needs have been carefully studied, and after deducting their present supply, the deficiencies have been assumed as being supplied from the Colorado River.

The cost of pumping this water, both annually and in total throughout the years, together with the interest charges and installments of principal that will have to be borne by the population to be served, has been computed as carefully as such data can be. The resulting figures are being applied to each proposed route as the field information becomes available.

Aqueducts and Dams

Various types of aqueduct construction are proposed. The quantity of water ultimately to be diverted is 1 500 cu. ft. per sec. When on hydraulic grade the line will be in excavation and completely covered throughout its length. When below hydraulic grade it will be either steel or reinforced concrete pipe, with possibly a pressure tunnel under San Gorgonio Pass. All tunnels will be lined with concrete.

The type of construction proposed for grade conduits is similar to that used on the Catskill Aqueduct. Where the nature of the ground surface makes it advisable to dip below the hydraulic grade and locate under light pressure, a reinforced concrete, hydro-static chord type similar to the 18-ft. conduit of the Ontario Power Company may be adopted. Under heavier pressure, steel pipe lines will be used. Due to the frequency of cloudbursts on the desert area to be traversed, the adoption of either an open conduit or a so-called cut-and-cover section blocking cross-drainage is not considered advisable. Pumping plants, pressure mains, and inverted siphons will be constructed in units as required.

Owing to the steepness of the slopes prevailing in the coastal plan of Southern California, there are few large reservoir sites available for regulating and distributing purposes at that end. The City of Pasadena has a proposed reservoir in San Gabriel Canyon, near Azusa, Calif., which, if constructed, would store 65 000 acre-ft. of water. The Los Angeles County Flood Control District also contemplates the construction of a reservoir near San Dimas, Calif., in what is known as the Puddingstone Reservoir site. It would be capable of storing at least 16 000 acre-ft., in addition to that required for flood-control purposes. Both these proposed reservoirs are situated at elevations which would make them practical for storage and distributing purposes, and all studies made thus far, contemplate maintaining the grade line at such elevation as to make use of either, if available.

Taking all these elements into consideration, the economics of the situation dictate that the route, gradients, and pumping lift to be adopted, be such as to place the least burden on the population to be served. Recognition must be given to the fact that the cities of Southern California are now growing rapidly, and will continue to grow at a rate which may be even faster than at present.

Power Requirements

In conclusion, the construction of any non-gravity aqueduct from the Colorado River to Los Angeles and neighboring cities, is predicated on the utilization of a large block of power. The only source which could furnish power in a sufficient quantity and at a cost not to be prohibitive, is the Colorado River. In order to generate this power, it would be necessary to construct a high dam on the river. The most favorable location for such a dam is either at Boulder Canyon or Black Canyon.

The investigations outlined in this paper have been made under the general supervision of William Mulholland, M. Am. Soc. C. E., Chief Engineer and General Manager, and the writer as Assistant Chief Engineer and General Manager of the Bureau of Water-Works and Supply of the City of Los Angeles. E. A. Bayley, M. Am. Soc. C. E., Assistant Engineer of this Bureau, has been in full charge of all field engineering.

TRANS-MOUNTAIN DIVERSIONS IN COLORADO

By Robert Follansbee,* M. Am. Soc. C. E.

Present Development

More than one-half the 103 658 sq. miles comprising the area of Colorado are within the Rocky Mountain region. The eastern part of the State lies within the province of the Great Plains, which extend from the front range of the Rockies to the Missouri River. Irrigation has been practiced in this region for many years, and the area of irrigable land is limited only by the water supply. The sources of this supply are the South Platte and Arkansas Rivers and their numerous tributaries, which rise on the eastern slope of the Rocky Mountains. With the present supply the limit of irrigation on the plains has almost been reached (except for the increased re-use of return seepage water).

In the mountainous region, except for certain relatively small areas, the water supply is in excess of that required by the irrigable lands. This situation has led water users to divert water from that region to the head-waters of the streams on the eastern slope. Table 1 summarizes the description of these diversions.

It will be noted from Table 1 that with the exception of Laramie-Poudre Tunnel which is 11 306 ft. long, all the diversions are made by relatively short open ditches at high elevations, and intercept the run-off from small areas.

Future Development

The eleven developments already made represent practically the limit of trans-mountain diversions by the relatively inexpensive open ditches, except for the proposed extension of Grand River Ditch. Additional developments must be of greater magnitude and expense, and be made by tunnels and collection ditches leading to them.

The need for additional water on the eastern slope has led water users to propose development from the Fraser, Williams, Blue, and Eagle Rivers and the Fryingpan Creek Basin, the only areas situated so that diversions from them to the Eastern slope can be seriously considered for many years to come. Of these, the chief are the Fraser and Blue River diversions. They are on a scale so large, that they can only be undertaken by a municipality like Denver, where the economic phase of the question is not so much a governing factor as the necessity for obtaining additional water to provide for the city's future needs.

It, therefore, appears entirely probable that when these projects are finally undertaken and carried to completion, the systems will be so planned and constructed that they will collect about 80% of the available water. In fact, the city's engineers charged with planning its future water supply are con-

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TABLE 1.—TRANS-MOUNTAIN DIVERSIONS IN COLORADO.

Ditch.	Elevation of diversion, in feet.	Drainage area intercepted, in square miles.	Diversion. From.	Diversion. To.	Conduit. Length of tunnel, in feet.	Conduit. Length of ditch, in miles.	Conduit. Capacity, in second-feet.	Annual diversion, in acre feet.
COLORADO RIVER BASIN.								
Grand River	10 200	12	Colorado River	Cache la Poudre River	0	8	220	11 400
Ewing	10 500	5	Eagle River	Arkansas River	0	5	20	1 910
Berthoud	10 300	4	Fraser River	Clear Creek	462	4	53	685
Cochetopa	10 000	20	Cochetopa Creek	Rio Grande	0	2	40	1 000
Boreas Pass	11 500	2	Blue River	South Platte River	0	1	10	600
NORTH PLATTE RIVER BASIN.								
Skyline	9 300	14	Laramie River	Cache la Poudre River	0	5	200	16 700
Sand Creek	10 000	...	" "	" " " "	0	10	200	2 900
Laramie-Poudre Tunnel	8 570	67	" "	" " " "	11 306	12.5	800	6 530
Bob Creek	9 800	2	" "	" " " "	0	2	......	840
Rist and McNab	10 300	4	Michigan River	" " " "	0	2.5	50	3 400
Cameron Pass	10 300	1	" "	" " " "	0	2	10	300
Total	......	...	......	......	11 768	54	1 603	46 300
Total for Colorado River	......	...	......	......	462	20	343	15 600

templating an efficiency of 90 per cent. To obtain this high efficiency, the collection ditches in earth and loose rock must be lined, and drainage ditches must be built along the upper side to collect the surface run-off and carry it to the cross-drainage channels which will empty into the main ditch. Also, provision must be made to clean the snow from the ditches by machines, early in the spring.

Judging from the experience gained from the operation of the Grand River and Skyline Ditches, it will be possible to exclude, during the winter months, the run-off from the intercepted streams entering the main ditches, and thus prevent the formation of any great quantity of ice. Also, it may be necessary to cover some parts of the ditches. On the capacities of the tunnels and ditches will depend the storage required. Ditches on south slopes will obtain water fully two weeks earlier than those on north slopes; the peak flow will be higher and its subsidence more rapid. Open ditches at lower elevations will encounter less slide rock and will be subject to less danger from heaving, due to the less severe frost action.

The period of the year for which diversion can be made, is taken as that from April 1 to September 30 because that is the average period during which the proposed diversions (except the relatively small one from Fryingpan Creek) can be made without interference with the right of the hydro-electric plant at Shoshone, on the Colorado River. In determining the available water supply no consideration has been given the conflict with water rights immediately below the proposed diversions, as these must be acquired and extinguished.

Fraser River.—With the completion of the Moffat Water Tunnel, which parallels the railroad tunnel at a distance of 75 ft., and has an elevation of 9 300 ft. at its mid-point, the City of Denver has planned to construct a system to divert water to the South Platte Basin. From the west portal of the water tunnel, in NE. ¼ Sec. 10, T. 25 N., R. 75 W., two collection ditches are to be constructed, one 27 miles long, reaching West St. Louis Creek, and the other, 9 miles long, reaching North Ranch Creek. This system will intercept the run-off from 107 sq. miles of drainage area, ranging in altitude from 9 300 to 12 500 ft. From the records of the Fraser River near West Portal (1911-26), the unit run-off for the period from April 1 to Sepember 30 is estimated as 1 060 acre-ft. per sq. mile, or 113 000 acre-ft. from the entire area intercepted. With an efficiency of 80% the mean annual diversion would be 90 400 acre-ft. A study of the 30-year record of the Colorado River near Palisades indicates that the mean for that period is 94% of the 16-year mean (1911-26). On this basis the mean annual diversion would be reduced to 85 000 acre-ft.

The annual variation in the proposed diversion is shown by Table 2, based on the annual percentages for Fraser River near West Portal, and the mean diversion of 90 400 acre-ft.

It will be noted from Table 2 that in the lowest year of the 16-year period, the diversion would have been 63 200 acre-ft.

To carry the maximum discharge of the wettest year, a tunnel capacity of 3 100 sec-ft. would be required. However, it is possible to obtain some

storage above the proposed diversions, and this would reduce the required tunnel capacity. A capacity of 1 000 sec-ft. would have required annual storage during the past 16 years, as follows:

Year.	Acre-feet.	Year.	Acre-feet.
1914	22 200	1921	15 400
1915	25 000	1923	3 880
1917	3 990	1924	3 990
1918	26 900	1926	3 590

TABLE 2.—Annual Diversion from Fraser River Basin.

Year.	Run-off above diversion points, in acre-feet.	Percentage of mean.
1911	83 000	92
1912	108 000	119
1913	66 800	74
1914	119 000	132
1915	108 000	119
1916	82 200	91
1917	84 100	93
1918	117 000	129
1919	63 200	70
1920	80 400	89
1921	106 000	117
1922	66 800	74
1923	86 800	96
1924	81 300	90
1925	75 900	84
1926	105 000	116
Mean	90 400	....

Williams River.—Several surveys have been made to investigate the diversion of water from Williams River to Clear Creek in the South Platte Basin. The most comprehensive survey has been made by the City of Denver, which proposes to bore a tunnel 3 miles long at an altitude of 10 300 ft.* The west portal will be on Bobtail Creek, a tributary of Williams River, and the east portal on the West Fork of Clear Creek. From the west portal, one collection ditch will extend to an unnamed creek beyond McQueary Creek, a distance of 4 miles, and another ditch will extend to the South Fork of the Williams River, a distance of 18 miles. These ditches will intercept the run-off from 29 sq. miles of drainage area, ranging in altitude from 10 400 to 12 500 ft. No records of the Upper Williams River are available, but by a comparison with the Fraser River records, the unit run-off above 10 400 ft. from April 1 to September 30, inclusive, is estimated as 1 100 acre-ft., or 32 000 acre-ft. for the entire 29 sq. miles. With an efficiency of 80%, the mean annual diversion would be 25 600 acre-ft. A tunnel capacity of 600 sec-ft. will be required without storage on the western slope to divert this quantity of water.

A study of the Palisades records indicates that the 30-year mean is 94% of the 16-year mean (1911-26). On this basis, the mean annual diversion would be reduced to 24 000 acre-ft.

* This is about the lowest altitude believed to be feasible for a water tunnel, as the slopes on both sides of the Continental Divide are such that a tunnel at 9 300 ft., the altitude of the Fraser River diversion, would be 11 miles long.

Blue River.—This basin is so situated with reference to the eastern slope that a number of possible diversions present themselves. Surveys have been made by the City of Denver for diversions at elevations of 10 300, 9 500, 9 112, and 8 842 ft.; and the writer has made a preliminary study for one at an elevation of 9 800 ft. Table 3 shows the possibilities at each elevation proposed.

TABLE 3.—PROPOSED DIVERSIONS FROM BLUE RIVER AT VARIOUS ELEVATIONS.

Diversion.		Elevation, in feet.	Length of Conduit.		Area intercepted, in square miles.	Mean annual diversion (80 %), in acre-feet.
From.	To.		Tunnel, in miles.	Ditch, in miles.		
Snake River.....	West Geneva Creek	10 300	3	26	56	28 000
Illinois Gulch...	Michigan Creek.........	9 800	7.5	25	100	57 000
Swan River......	Halls Gulch.............	9 500	12.9	26	181	96 000
Blue River	North Fork, South Platte River..........	9 112	18.9	14	293	189 000
" "	North Fork, South Platte River..........	8 842	22.8	1	328	197 000

The quantity of water that can be diverted will depend on the length of the collection ditches built from the receiving portal of the tunnel along the mountain sides. Since the diversion of a given quantity of water requires a shorter collection ditch as the elevation of the tunnel is decreased, it follows that, for the same length of collection ditch, the quantity of water that can be diverted, will increase with the decrease in elevation.

In Table 3 it will be noted that for 26 miles of ditches the diversions would increase from 28 000 to 96 000 acre-ft., with an increase in tunnel length from 3 to 12.9 miles. The projects requiring tunnel lengths of 18.9 and 22.8 miles make available such large diversions that a study showing those available with ditches 26 miles long has not been made. Since the City of Denver is the only water user that can afford to make a diversion from Blue River, at least within the period of time for which a forecast of future use can safely be made, and since a tunnel 12.9 miles long will yield a quantity of water sufficient for Denver's needs for the same period, and is within the realm of present-day engineering practice, the diversion of 96 000 acre-ft. at an elevation of 9 500 ft. has been selected for the purpose of this paper.

The west portal would probably be on Swan River, in SE. ¼ Sec. 15, T. 6 S., R. 77 W., 1 mile below Tiger. Collection ditches extending from the North Fork of the Snake River on the north to North Barton Creek on the south would have a total length of 26 miles and intercept the run-off from 181 sq. miles of drainage area ranging in elevation from 9 500 to 13 500 ft. Stream-gauging records are available for the Blue River, at Dillon (1911–26) and for the Snake River, at Dillon (1911-19). For the nine-year period (1911-19), the mean run-off from April 1 to September 30 is 148 000 acre-ft., or 676 acre-ft. per sq. mile for 219 sq. miles, ranging in elevation from 8 850 to 13 500 ft. As the run-off increases with elevation, the unit run-off for the

area above 9 500 ft. will be slightly greater, and has been taken as 718 acre-ft. This gives a total run-off of 120 000 acre-ft. above the 181 sq. miles lying above the collection ditches. With an efficiency of 80%, the mean annual diversion will be 96 000 acre-ft.

A study of the 30-year record of the Colorado River near Palisades indicates that the mean for that period is 95% of that for the 9-year period covered by the Blue and Snake River records. On this basis the mean annual diversion from Blue River would be 91 200 acre-ft.

The variation in run-off available for diversion from April 1 to September 30 each year under the project, at 9 500 ft. elevation, is shown by Table 4 which is based on a mean run-off of 96 000 acre-ft., and an annual percentage determined from the run-off from April to September each year at the gauging station on Blue River, at Dillon.

TABLE 4.—ANNUAL DIVERSION FROM BLUE RIVER BASIN.

Year.	Run-off above diversion points, in acre-feet.	Percentage of mean.
1911	82 500	86
1912	115 000	120
1913	86 500	90
1914	134 000	140
1915	82 500	86
1916	85 500	89
1917	106 000	111
1918	111 000	116
1919	69 000	72
1920	87 300	91
1921	121 000	126
1922	72 000	75
1923	111 000	116
1924	87 200	91
1925	72 000	75
1926	119 000	124
Mean	96 000	....

It will be noted that in the lowest year of the 16-year period, the diversion would have been 69 000 acre-ft.

The maximum discharge at the tunnel site during the years of record was 1 880 sec-ft. With a tunnel capacity of 1 000 sec-ft., the following storage would have been required to care for the excess run-off:

Year.	Acre-feet.
1914	19 500
1917	6 480
1918	17 000
1921	10 700
1923	2 050
1924	1 930
1926	6 740

Eagle River.—At Tennessee Pass, which separates the Eagle and Arkansas Drainage Basins, an opportunity is afforded for the diversion of water from Eagle River. A tunnel at an elevation of 10 200 ft. would be 2 miles long

and at 9 800 ft., 6 miles long. Since the slope of Eagle River Basin prevents the interception of any area comparable in size with the areas in the Fraser and Blue Drainage Basins, it is evident that for the quantity of water available, a length of tunnel greater than 2 miles would not be warranted. With the tunnel at 10 200 ft. elevation and 15 miles of collection ditches reaching Bennett Creek on the west side, and East Fork on the east, an area of 30 sq. miles will be intercepted, ranging in elevation from 10 200 ft. to 13 000 ft. Records of Eagle River at Redcliffe from 1911 to 1925 show the mean run-off from April 1 to September 30 to be 594 acre-ft. per sq. mile. For the area above 10 200 ft. elevation, the unit run-off is estimated as 700 acre-ft. per sq. mile, or 21 000 acre-ft. for the 30 sq. miles intercepted. With an operating efficiency of 80%, 16 800 acre-ft. could be diverted.

A low divide between Eagle River and Tenmile Creek Drainage Basins makes possible a diversion at 10 900 ft. by a tunnel ¼ mile long.* A ditch, 3½ miles long, on the west side reaching Searles Gulch, and one 6½ miles long on the east side, reaching Mayflower Gulch, will intercept an area of 20 sq. miles, ranging in elevation from 10 900 to 13 500 ft. Records of Tenmile Creek, at Dillon, from 1911 to 1919, show the mean run-off from April 1 to September 30 to be 872 acre-ft. per sq. mile. For the area above 10 900 ft., the unit run-off is estimated as 1 000 acre-ft. per sq. mile, or 20 000 acre-ft. for the entire area intercepted. With an efficiency of 80%, 16 000 acre-ft. could be diverted. The diversion from Tenmile Creek would be made to the East Side Collection Ditch of the Eagle River System.

The annual variation in the combined diversions proposed from the Eagle and Tenmile Basins is shown in Table 5, which is based on a mean discharge of 32 800 acre-ft. adjusted according to the variation at the Eagle and Tenmile Gauging Stations.

TABLE 5.—ANNUAL DIVERSION FROM EAGLE RIVER AND TENMILE CREEK.

Year.	Run-off above diversion points, in acre-feet.	Percentage of mean.
1911	39 000	119
1912	42 600	130
1913	25 600	78
1914	41 300	126
1915	23 300	71
1916	31 800	97
1917	33 800	103
1918	41 300	126
1919	24 300	74
1920	33 400	102
1921	41 300	126
1922	23 000	70
1923	34 400	105
1924	27 600	84
1925	22 700	69
Mean	32 800	.....

It will be noted in Table 5 that in the lowest year of the 15-year period, the diversion would have been 22 700 acre-ft. Without storage on the western slope, a tunnel capacity of 700 sec-ft. would be required.

* Lowering the elevation of the tunnel 100 ft. would increase its length to 1½ miles.

From the Palisades records it appears that the 29-year mean (1897-1925) is 93% of the 15-year mean (1911-1925), and, on this basis, the mean run-off available for diversion would be 30 500 acre-ft.

Fryingpan Creek.—Ivanhoe Tunnel, in Sec. 13, T. 9 S., R. 82 W., 11 miles west of Leadville, on the abandoned Colorado Midland Railway, affords an opportunity to divert water from Fryingpan Creek to Lake Fork, a tributary of the Arkansas River. The tunnel, which is at an altitude of 10 950 ft., is 2 miles long, with an eastward slope of 15 ft. per 1 000. Near the west portal, lies Lake Ivanhoe which can be converted into a reservoir of 1 920 acre-ft. capacity by a 30-ft. dam at each end. A collection ditch, 14 miles long, heading in the main fork of Fryingpan Creek, would intercept the run-off from 12 sq. miles of drainage area, ranging in altitude from 11 000 ft. to 13 500 ft. Records of Fryingpan Creek at Norrie (Elevation 8 440), from 1911 to 1916, show the mean run-off from April 1 to September 30 to be 1 040 acre-ft. per sq. mile. For the area above 11 000 ft., the unit run-off is estimated to be 1 300 acre-ft. per sq. mile, or 16 000 acre-ft. for the 12 sq. miles intercepted. At an efficiency of 80% the annual diversion would be 12 800 acre-ft. Irrigation interests in the Arkansas Valley have started construction work on this diversion.

Summary of Future Development.—The developments here described are summarized in Table 6.

TABLE 6.—SUMMARY OF FUTURE DEVELOPMENT.

DIVERSION.		Drainage area intercepted, in square miles.	Elevation of tunnel, in feet.	LENGTH OF CONDUIT, IN MILES.		Annual diversion, in acre-feet.
From.	To.			Tunnel.	Ditch.	
Fraser River........	South Boulder Creek.	107	9 300	6	36	90 400
Williams River.....	West Clear Creek.....	29	10 300	3	22	25 600
Blue River..........	South Platte River....	181	9 500	12.9	26	96 000
Eagle River.........	Arkansas River........	50*	10 200	2	15	32 800
Fryingpan Creek...	" "	12	10 950	2†	14	12 800
Total............	......	379	......	25.9	113	258 000

* Including diversion from Tenmile Creek.

† Ivanhoe Tunnel on abandoned Colorado Midland Railway.

From the magnitude of the work involved, it will probably be many years before these diversions are completely developed.

Effect of Diversions on the Flow of the Colorado River.—To determine the effect of the possible diversions on the flow of the Colorado River at various points, it is necessary to compute the monthly quantities to be diverted. For that purpose, the period, 1911-26, in so far as available for each stream, was selected, and the monthly percentages for the entire discharge of the period from April 1 to September 30 were determined from the records on each stream. These percentages were applied to the mean diversions for each project.

TABLE 7.—MEAN MONTHLY DISCHARGE OF PROPOSED DIVERSIONS.

Drainage basin.	Discharge, in Acre-Feet.					
	April.	May.	June.	July.	August.	September.
Fraser River...........	2 710	17 200	40 700	18 100	7 220	4 520
Williams River.........	1 020	4 850	11 000	5 370	2 050	1 280
Blue River.............	3 840	19 200	36 500	21 100	9 600	5 760
Eagle River............	1 970	9 520	12 500	5 240	2 290	1 300
Fryingpan Creek	640	3 070	5 380	2 300	900	510
Total	10 200	53 800	106 000	52 100	22 100	13 400

The effect of these diversions on the flow of the Colorado River is shown by deducting from the mean monthly recorded flow for the period, 1911 to 1926, in so far as it is available, the computed quantities given in Table 7, and expressing the differences as percentages of the recorded flow. These are given in Table 8 for each principal gauging station above the Green River, and for Yuma, Ariz.

TABLE 8.—DISCHARGE MODIFIED BY PROPOSED DIVERSIONS EXPRESSED AS PERCENTAGES OF MEASURED DISCHARGE.

Gauging station.	April.	May.	June.	July.	August.	September.
Kremmling, Colo.*	91	85	82	80	78	77
Glenwood Springs, Colo.†.	93	90	88	85	84	85
Palisades. Colo ‡..........	96	94	93	91	91	92
Cisco, Utah..............	98	97	95	94	94	94
Yuma, Ariz...............	99.3	98.0	97.8	98.0	98.0	97.6

* Above diversions from Eagle River and Fryingpan Creek.
† Above diversions from Fryingpan Creek.
‡ Above diversions for Grand Valley.

The effect of the proposed diversions on the flow of the Colorado River will be a maximum at the Kremmling Station and will decrease at points farther down stream where the discharge is successively greater. From the substantial uniformity of annual variation in discharge throughout the Upper Basin, it is believed that the percentages in Table 8 will apply each year.

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PAPERS AND DISCUSSIONS

THE EYE-BAR CABLE SUSPENSION BRIDGE AT FLORIANOPOLIS, BRAZIL

Discussion*

By D. B. Steinman and William G. Grove, Members, Am. Soc. C. E.†

D. B. Steinman‡ and William G. Grove,§ Members, Am. Soc. C. E. (by letter).||—The economic advantages of the Florianopolis type of suspension bridge received commercial recognition and confirmation in the recent proposal and adoption of this form of construction in two successive bidding competitions. These are the 700-ft. spans over the Ohio River at Point Pleasant, W. Va., and St. Mary's, W. Va., respectively, now under construction. In each case, the Florianopolis type was proposed by the American Bridge Company as an alternative bid, at a price sufficiently below the tenders on the conventional suspension designs to win the respective competitions. The bids showed that, for a span length of 700 ft. and for the relative unit stresses specified, heat-treated eye-bars are more economical than wire cables at present prices; and that the combination of the eye-bars with the Florianopolis type offers a further saving in the cost of the structure.

It is true, as stated in some of the discussions, that the relative economy of eye-bars and wire cables was not conclusively demonstrated by the case of the Florianopolis Bridge. The relative unit stresses specified for the two materials and the length of the span are governing factors. In the wire cable design for Florianopolis, the specified working stress was 70 000 lb. per sq. in., and the heat-treated eye-bars were offered (at equal total cost) to be used with a working stress of 50 000 lb. per sq. in. Had a higher stress been specified for the wire or a lower stress for the eye-bars, so as to provide equal factors of safety for the two materials, it would have been more difficult for

* Discussion of the paper by D. B. Steinman and William G. Grove, Members, Am. Soc. C. E., continued from February, 1928, *Proceedings*.

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|| Received by the Secretary, February 16, 1928.

the eye-bars to be offered at equal cost. In the opinion of the designers of the Florianopolis Bridge, the fair stress ratio to use is 2 to 1, since that is approximately the ratio of the respective specified yield points or ultimate strengths of the two materials. This suggested ratio of working stresses has not yet become established because of the difficulty of overcoming the conventions of past conservative practice in wire cable design—a handicap from which the newer material of heat-treated eye-bars does not suffer.

There is, however, a marked present trend toward the adoption of higher unit stresses for cable wire. For the Ohio River Bridge, at Portsmouth, Ohio, the adopted design stress for the wire cables was 80 000 lb. per sq. in. In the recent bidding competition for the Hudson River Bridge, at New York, N. Y., the respective specified unit stresses were 50 000 lb. per sq. in. (with the secondary stresses included) for the heat-treated eye-bars, and 82 000 lb. per sq. in. (with secondary stresses not included) for the wire cables. During the past few months, heat-treated cable wire has been developed, offering a minimum yield point of 190 000 and a minimum ultimate strength of 220 000 lb. per sq. in. This wire has been adopted for the international bridge at Detroit, Mich. (span, 1 850 ft.) and for the Mount Hope Bay Bridge, in Rhode Island (span, 1 200 ft.), with a working stress of 84 000 lb. per sq. in. for the former and 88 000 lb. per sq. in. for the latter. Heat-treated bright wire is now offered with a yield point of 200 000 and an ultimate strength of 240 000 lb. per sq. in., for which still higher unit stresses would be specified.

In the bidding on the Hudson River Bridge, the wire design won by a margin of \$2 000 000, indicating that the eye-bars cannot compete with wire cables for extremely long spans. The awards on the Point Pleasant and St. Mary's Bridges indicate that heat-treated eye-bars, at present relative prices and working stresses, are more economical than wire cables for spans as short as 700 ft. It is difficult to predict the relative economy of the two materials for span lengths between these limits (700 ft. and 3 500 ft.) without further bidding competitions. Any conclusions are subject to revision with changes in relative working stresses and with fluctuations in unit prices.

Although the manufacture of eye-bars is now a monopoly, the prices for heat-treated eye-bars are affected by the economic competition offered by wire cables. This competition, which came into play with the bidding on the Florianopolis Bridge following the development of the heat-treated eye-bars, has had a wholesome effect in stimulating improvements in the materials for higher physical properties and improvements in economic production toward reduced unit prices; also in prompting engineers to scrutinize established conventions with a view toward securing a more consistent basis for the specification of working stresses.

The writers fully appreciate the generous discussions that have been contributed to this paper, and will refer to them in the order of their presentation.

Mr. Waddell raises some questions in regard to details of proportions and form for designs of the Florianopolis type.* The writers' statement† that the

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1307.

† *Loc. cit.*, May, 1927, Papers and Discussions, p. 707.

utilization of the cable as the upper chord of the stiffening truss "should preferably be limited to the central half of the span", referred only to the loss of economy if the construction were extended to the ends of the span; it was not intended to specify any definite proportion as most economical. A variation of a few panels on either side of the quarter-points would produce a negligible difference in economy of material; practical considerations govern instead. In the case of the Florianopolis Bridge, after several variations of the design were studied, the outline shown in Fig. 2 (*a*)* was adopted, with somewhat less than one-half the span (11 double panels out of 27) utilizing the cable as the top chord; this was done to avoid excessive height, and excessive variation in height, of the truss. In the design adopted for the Ohio River Bridges at Point Pleasant and St. Mary's, the central construction also extends over less than the half-span (12 panels out of 28), on the basis of similar practical considerations.

The proportions and outlines suggested by Mr. Waddell were included among those studied for the Florianopolis Bridge and were abandoned in favor of the adopted layout. The question of the comparative appearance of the design used and that suggested by Mr. Waddell in his Fig. 38† is one of individual taste; this also applies to his remark that the conventional design, Fig. 2 (*b*),* would have had a better appearance with a deeper truss and with fewer suspenders; on both these points there are many who would take the opposing view. In the adopted design, the top chords in the outer quarters of the span were made straight, rather than curved, for the advantages of economy and simplicity in detailing and fabrication, as well as for structural efficiency. It is questionable whether curved top chords in the outer quarters (Fig. 38) really offer improved appearance, except where the construction is carried symmetrically into the side spans as in Fig. 42.‡

Mr. Waddell, while recognizing the justification of the claims for greater rigidity of the new type of stiffening truss, inquires in regard to confirmatory deflection tests on the finished structure. Such tests cannot be made without great difficulty until the railroad, which was intended to traverse the bridge, is built; and it is doubtful whether the local governmental authorities would be sufficiently interested in the scientific value of the results to authorize the expense of deflection tests. The designers are confident of the confirmation that would be yielded by such tests, for it is invariably found that the actual measured deflections of a suspension bridge are less than the calculated amounts. Although no live load deflection tests have been recorded for the Florianopolis Bridge, a close verification of the computed dead load deflections was afforded by the official test to determine whether the required vertical under-clearance from water level had been realized.

The problem of supporting the rocker towers during their construction and during the erection of the eye-bar cables was given considerable study. A solution was found whereby the actual amount of erection material required

* *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 710.

† *Loc. cit.*, August, 1927, Papers and Discussions, p. 1308.

‡ *Loc. cit.*, December, 1927, Papers and Discussions, p. 2703.

was very small, consisting merely of some plates and angles at the ends of the temporary struts which tied the tower columns to the U-1 and L-1 points of the adjoining 185-ft. spans. The struts themselves consisted of material later used for the suspension-span bracing in the finished structure.

The principle of the double-bored pin-hole in the eye-bar heads was used by the American Bridge Company in 1912 in the connecting links joining the spans of the Kenova Bridge* during cantilever erection, in order to facilitate the swinging of the spans. To the writers' best knowledge, this principle had never actually been incorporated in holes in the heads of eye-bars, prior to its use on the Florianopolis Bridge.

The writers appreciate the complimentary remarks† of Mr. Miller, relative to the use of 1-in. ropes to form the temporary cable from which the eye-bars were erected.

The twenty-four erection ropes were grouped together in the form of an elongated hexagon shown in Fig. 43 (a). The wheels of the main-span trolley used to erect the eye-bar cables were made to fit this hexagon, so that eight ropes were actually in contact with the wheels. They were closely compacted to the remaining sixteen ropes by the weight of the trolleys and by the loads of eye-bars applied to the ropes by the chain hoists.

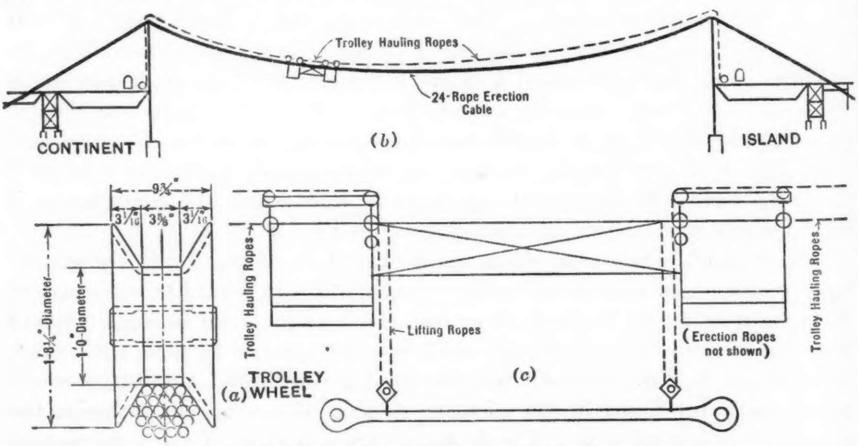

FIG. 43.—MAIN SPAN EYE-BAR TROLLEY OPERATION.

After the pilot rope was erected, the remaining twenty-three ropes were placed and adjusted to the same sag as the pilot rope. Each rope under its own weight then had a stress of approximately 3 000 lb. and had stretched about 2.7 ft. in 1 700 ft. Later, when the entire weight of the eye-bars was supported by the ropes, each rope had an average stress of approximately 30 000 lb. and had stretched about 15.1 ft. in 1 700 ft.

The actual elongations of the nine test pieces of rope, when stressed the first time to 30 000 lb., are given in Column (2) of Table 15. The actual

* See "Reconstruction of the Norfolk and Western Railway Company's Bridge Over the Ohio River at Kenova, West Virginia," by William G. Grove, M. Am. Soc. C. E., and Henry Taylor, Esq., *Transactions*, Am. Soc. C. E., Vol. LXXIX (1915), p. 411.

† *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1309.

elongations of Ropes *L, M, R, S* and *X,* when stressed the second time to 30 000 lb., are given in Column (4). The average elongation for Column (2) is 0.0895 in. in 10 in., very close to the elongation of Test Rope *M.* For Column (4) the average elongation is 0.0891 in. in 10 in., very close to the elongations of Test Ropes *M* and *X,* or practically 0.89 per cent.

TABLE 15.—ELONGATION AND STRESSES IN TEST PIECES OF ROPE.

	FIRST APPLICATION OF LOAD.		SECOND APPLICATION OF LOAD.	
	Elongation, in 10 in.	Imaginary cable stress, in pounds.	Elongation, in 10 in.	Imaginary cable stress, in pounds.
(1)	(2)	(3)	(4)	(5)
Rope *A*	0.0950	28 300	......	......
" *F*	0.0905	29 700	.....	......
" *G*	0.0997	27 000	......	......
" *K*	0.0976	27 500	......	......
" *L*	0.0984	27 300	0.1042	25 600
" *M*	0.0897	30 000	0.0884	30 200
" *R*	0.0657	41 000	0.0712	37 600
" *S*	0.0850	31 600	0.0929	28 800
" *X*	0.0841	32 000	0.0886	30 200
Average	0.0895	30 500	0.0891	30 500

Assuming a composite cable made up of the nine test ropes to carry an average of 30 000 lb. per rope when elongated 0.89%, it is found that Test Rope *R* would carry more, and Test Rope *L* less, than 30 000 lb. The actual stress that each test rope would carry when the 9-rope cable is stressed to 30 000 lb. the first time is given in Column (3) of Table 15 and the actual stress that each of Test Ropes, *L, M, R, S,* and *X* would carry when this rope cable is stressed to 30 000 lb. the second time is given in Column (5). It is seen that the actual stress in the nine test ropes would vary from 27 300 to 41 000 lb. when the 9-rope cable is stressed to 30 000 lb. the first time, and the actual stress in the five test ropes would vary from 25 600 to 37 600 lb. when the 5-rope cable is stressed to 30 000 lb. the second time. From this analysis it may be reasonable to assume that the actual stress in the twenty-four ropes used in the cable varied from 25 000 to 40 000 lb., when the entire 24-rope cable carried an average stress of 30 000 lb. per rope.

In reference to the operating ropes for the main-span trolley used to erect the eye-bars, Fig. 43 (*b*) is a diagrammatic view of the operating lines running to the tops of the towers and thence down to engines on the viaduct floor. Fig. 43 (*c*) shows, to enlarged scale, both the operating and the load lines. When the continent engine pulled the trolley westward, the island engine played out the operating rope. When the trolley had been properly spotted, both engines were used to raise the eye-bars. Either end of the eye-bars could be raised or lowered slightly by operating the particular engine controlling that end of the bar without operating the other engine.

Fig. 44 shows diagrammatically the trolley used for the erection of the suspended span. In the upper view (Fig. 44 (*a*)) are seen the operating lines to the two engines which controlled the movements of this trolley in a manner similar to that in which the movements of the main-span eye-bar trolley were controlled. The lower view (Fig. 44 (*b*)) shows both operating and load lines. In the case of the load line, there was only one set of falls and the line ran continuous from engine to engine. When the lift was near the continent, the continent engine was operated; when near the island, the control was by the island engine. When the trolley itself was moved, the lifting line ran idle through the block.

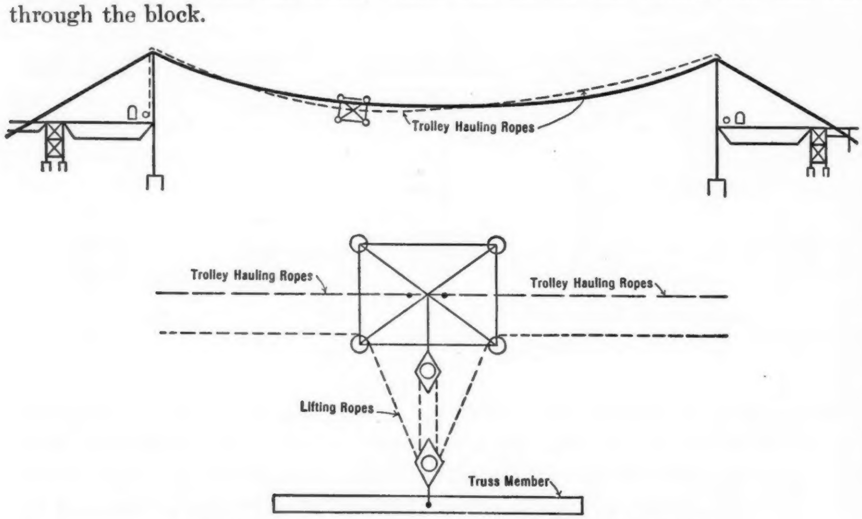

FIG. 44.—TROLLEY USED FOR SUSPENDED SPAN ERECTION.

Mr. Frost states* that the Florianopolis design and its method of erection may extend the range of use of structures of the suspension type. In this connection, it may be noted that the past few years have marked a period of renewed activity all over the United States in the design and construction of suspension bridges, especially for spans ranging in length from 600 to 1 200 ft. Contributing factors have been improvements in economical design and erection, the development of higher strength eye-bars and wire, and the competitive reduction of unit prices on the two materials. Erroneous notions of the relative expensiveness of suspension construction have been partly dispelled, and the advantages of the type have become more widely recognized. It is difficult, at this stage, to appraise the influence of the Florianopolis design and method of erection on this development. Nevertheless, it may be significant to note that four Ohio River bridges (Portsmouth, Steubenville, Point Pleasant, and St. Mary's) commenced in 1926-27, are suspension structures, where cantilever construction previously prevailed; and that only two of these bridges are wire cable designs, the other two being eye-bar designs of the Florianopolis type.

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1310.

Mr. Larsson presents* an interesting historical outline of the development of the eye-bar industry. As is generally well known in the profession, Mr. Larsson with the late C. W. Bryan, M. Am. Soc. C. E., Chief Engineer of the American Bridge Company, deserves full credit for the development of the heat-treated eye-bars, first in the medium grade (elastic limit, 50 000 lb. per sq. in.) developed in 1914, and, subsequently, in the high-tension grade (elastic limit, 75 000 lb. per sq. in.) developed in 1921 and receiving its first practical application in the Florianopolis Bridge. It was this high-tension grade of heat-treated eye-bars that made it possible for an eye-bar design to be proposed for competitive bidding as one of the two alternative official designs for the 3 500-ft. Hudson River Bridge.

A deterrent to the more ready acceptance of the new material has been the policy of secrecy covering its manufacture. Structural engineers hesitate to accept a material without full privileges of inspection and chemical analysis. Mr. Larsson's present announcement that the policy of secrecy has been discontinued, will be welcomed by the profession. The remaining deterrent, namely, the difficulty of making full-sized tests, should next be removed by the provision of a testing machine large enough to take the longest bars furnished for any structure.

Professor Morris reports† an eye-bar suspension bridge of 450-ft. span built in 1914-15 over the Muskingum River, at Dresden, Ohio. The writers regret that they did not know of this structure and that Professor Morris had therein preceded them in the adoption of rocker towers. It is also of interest to note that for this span of 450 ft., constructed in 1914-15, eye-bars of structural steel grade (ultimate strength, 60 000 to 70 000 lb. per sq. in.) won over wire cables in the bidding competition.

An instructive joint discussion‡ has been submitted by Messrs. Covell, Wilkerson, and Nutter. They commend the Florianopolis Bridge as "a splendid type" from the standpoint of economy, rigidity, and general structural efficiency. On the question of its esthetic value, they express a preference for a curved side-span design with symmetrical construction, like Fig. 42§; but such design would not have suited the local conditions at Florianopolis. On the question of economy, they point out that the total weight of steel in the Florianopolis Bridge is surprisingly low for a structure with a main span of more than 1 100 ft. and a total length of about 2 700 ft.; and the credit for this showing of economy they rightly ascribe to the type of span and the use of high-tension heat-treated eye-bars and a timber floor. On the last point, it may be stated that advantage was taken of the availability of South American hardwood to provide a floor material of great strength and longevity.

The maximum loadings of 50-ton locomotives for the railway and 6-ton trucks for the highway were the official Government specifications for the

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1310.

† *Loc. cit.*, p. 1312.

‡ *Loc. cit.*, September, 1927, Papers and Discussions, p. 1741.

§ *Loc. cit.*, December, 1927, Papers and Discussions, p. 2703.

design; and the local authorities regard as extremely remote any possibility of these loadings ever being exceeded in that locality.

Messrs. Covell, Wilkerson, and Nutter agree that the high-tension heat-treated eye-bars are a proper material for an eye-bar chain (as in the Florianopolis Bridge), but they question whether the ductility of 5% would be sufficient for the use of this material for truss members. To the writers' knowledge, the high-tension eye-bars have not thus far been used in any truss structures; only the medium grade heat-treated eye-bars (8% elongation) have been so used, as in the Carquinez Cantilever Bridge, with two spans of 1 100 ft., and in the Oil City simple spans of 260 and 350 ft.

The three engineers also agree with the writers that the hinged tower for the suspension bridge is a step in the right direction, and they cite their adoption of the rocker-tower design for the Sixth, Seventh, and Ninth Street Suspension Bridges over the Allegheny River at Pittsburgh, Pa.

With respect to appearance, it is impossible to get a real profile view of the Florianopolis structure except when crossing the Strait. From any point on either shore, the bridge is viewed obliquely, and the eye-bar cable seems larger due to seeing all four eye-bars of each cable. The photograph, Fig. 3*, does not do justice to the appearance of the structure as ordinarily viewed.

The writers desire to thank Mr. Carstarphen for his comprehensive discussion† of the erection problem with reference to the behavior of the wire ropes used to erect the eye-bar cables.

They agree with him that two applications of the load are not sufficient to determine the true value of the modulus of elasticity of wire rope. In the use of the ropes at Florianopolis, however, the modulus of elasticity was not of direct interest, except as a means for determining the actual elongations of the ropes under certain conditions of loading during the erection. There were only two eye-bar cables to erect so that there was an actual condition in the field corresponding to Mr. Carstarphen's Tests (*a*) and (*b*) of Fig. 41‡. For these requirements, it was necessary to compute the elongations starting from the original length of the rope, and not from a permanent set value determined from a previous stress in the rope. Referring to Fig. 41, if the modulus was based upon an elongation starting not from the permanent set points, but from the zero points, the result would be as shown in Table 16. The resulting average value of the modulus is fairly close to 8 300 000 lb. per sq. in., the value used at Florianopolis.

As Mr. Carstarphen states, it is known from actual practice that hemp center ropes do not fall apart when bent around sheaves. In fact, the American Bridge Company uses 16-in. sheaves as standard sheaves with 1-in. rope.

In Mr. Carstarphen's discussion§, he arrives at a horizontal component in the erection ropes (messenger cables), when these ropes supported all the eye-

* *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 713.

† *Loc. cit.*, October, 1927, Papers and Discussions, p. 2031.

‡ *Loc. cit.*, p. 2032.

§ *Loc. cit.*, pp. 2034 and 2036.

bars, of 595 627 lb. instead of 660 000 lb. mentioned in the paper. To obtain the horizontal component of 519 527 lb. in the eye-bar cable under its own weight, with a sag of 116 ft., he used the dead load panel concentrations for the eye-bar cable only, as given in Table 10*. It was the original intention to erect not only the eye-bar cable, but also the permanent rope hangers (at the ends) and the top chord gusset-plates (at the central portion of the span) from the erection ropes; and the 116-ft. sag was based on the eye-bar cable plus this additional weight. Considering these additional loads, the horizontal component in the eye-bar cable would be 569 000 lb. instead of 519 527 lb. He also assumes 616 lb. for the weight of the chain hoists and their connections; this latter figure was actually very close to 1 000 lb. These additional weights on the erection ropes account for the difference between the two calculated values of 660 000 lb. and 595 627 lb. for the horizontal component in the ropes.

TABLE 16.—ELONGATIONS IN TEST SPECIMENS.

Reference.	Load, in pounds.	Percentage of elongation in 10 in.
Fig. 41 (*c*)	158 400	0.451
" (*d*)	158 400	0.436
" (*e*)	158 000	0.428
" (*f*)	156 600	0.420
Average	157 850	0.434

$$E = \frac{157\,850 \times 10}{2.4\,[0.0434 + 0.0410]} = 7\,800\,000$$

The length of the erection rope cable was based on the assumption of a parabolic curve, but the formula was extended to include the third term in the series,

$$L = l + \frac{8}{3}\frac{y^2}{l} - \frac{32}{5}\frac{y^4}{l^3} + \ldots.$$

it having been found that this last term had an appreciable influence on the length.

Mr. Hoadley reports† an independent stress analysis which he made of the Florianopolis Bridge for thesis work. He presents an outline of his method, using the "least work" equation based on Castigliano's principle, and he shows that it is merely another form of expressing the influence line equation for the method of elastic weights as used by the designers of the bridge. The values derived by Mr. Hoadley for the H-influence line are somewhat higher than those of Messrs. Robinson and Steinman (5% higher at mid-span), and he volunteers the explanation that this may be due to an inherent error in his method. His treatment of the top chord members in the middle part of the span requires him to add the contributions of the opposing tension and compression components arithmetically, whereas they should be added

* *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 750.

† *Loc. cit.*, October, 1927, Papers and Discussions, p. 2039.

algebraically. A review of Mr. Hoadley's thesis shows that his resultant values for the governing stresses in the stiffening truss are lower than the values obtained by Messrs. Robinson and Steinman and used by them in proportioning the structure, affording a welcome independent check on the safety of the design analysis. Mr. Hoadley also derives the H-influence line equation by a modified method, and computes therefrom a value of H agreeing with that found by Messrs. Robinson and Steinman by the method of elastic weights.

Professor Menefee states* that there is no longer any real mystery about heat treatment, and he predicts that all the large companies will soon be producing heat-treated bridge members as wanted. He approves the use of the Brinell test as a check on the uniformity of the heat treatment and of the tensile strength. Referring to the absence of the usual secondary and impact stresses, he considers 46 500 lb. per sq. in. (as used in the Florianopolis eye-bar cables) a safe working stress for steel with an elastic limit of 80 000 lb. per sq. in.

The writers agree with Professor Menefee that the substitution of eye-bars for wire cables adds stiffness and reduces deflections. His statement that a cable of equal sectional area will elongate under load more than a solid piece of steel requires clarification. For the same load and the same area of cross-section, a parallel wire cable and a chain of heat-treated eye-bars should show substantially the same elongation, since the respective values of the modulus of elasticity are substantially equal (about 27 000 000 lb. per sq. in.). The more recently developed heat-treated cable wire has a higher value of E (29 000 000 lb. per sq. in.), and, therefore, would elongate about 7% less than heat-treated eye-bars. The reason an eye-bar chain as applied in an actual design would yield smaller elastic elongation and resulting deflection than a parallel wire cable is because the eye-bar chain would be designed with a lower working stress and, therefore, would be provided with a proportionally larger sectional area. With the incorporation of the Florianopolis type, this increase in rigidity is further augmented (very materially) by the effect of the modified form of stiffening truss.

Mr. Kuo presents† a valuable discussion of the inherent problem of providing rigidity in suspension bridges. He distinguishes between free cable bridges (with suspended truss) and braced (triangulated) cable types; and he points out that to secure high rigidity in a free cable suspension bridge requires an unusually heavy stiffening truss, resulting in an uneconomical design of clumsy appearance. He considers the Florianopolis Bridge (which he classifies as a partly braced type) a very scientific and economical solution of the problem.

To the practical reasons given in the paper for seeking rigidity, Mr. Kuo adds another of more technical nature, namely, the reduction of forced deformations in the floor system. He reports having calculated extremely high secondary stresses in stringers resulting from stiffening truss deflections.

In regard to the use of rocker towers, Mr. Kuo points out a possible objection, in some locations, on the score of reduced security against severe earth-

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2327.

† *Loc. cit.*, p. 2328.

quake effects. The dowels provided in the base castings of the Florianopolis towers are intended to provide resistance to displacement by such forces; and the writers believe that, with proper design of the base details, rocker towers can be given the same security as fixed base towers against earthquake forces.

Mr. Kuo commends the new departure of the stiffening truss from parallel chords and the unification of the cable with the middle portion of the top chord as the two most admirable features of the Florianopolis design, representing a distinct development in the art of suspension bridge construction. He also points out the visible structural fitness of a design in which the truss depth varies with the magnitude of the bending moments.

Mr. Moisseiff recounts* the evolution of stiffening truss forms to that used at Florianopolis. On this subject, to facilitate a review of structural and chronological relationship, the writers have prepared Fig. 45, showing successive developments of suspension bridge forms, illustrated as far as possible with actual applications and their dates. The development of braced cable types embraces the form with full trussing between cable and roadway (like the bridge at Frankfort-on-Main, Germany, 1869-1921, Fig. 45(*c*)); the three-hinged form (Fig. 45(*d*)), with parabolic lower chord and straight top chords (represented by Hemberle's Point Bridge at Pittsburgh, 1877-1927); the forms with parallel chords (like the Seventh Street Bridge, at Pittsburgh, 1884-1926, Fig. 45(*e*), designed by Gustav Lindenthal, M. Am. Soc. C. E., and his more recent design for a Hudson River Bridge at 59th Street, New York); the Fidler truss and other three-hinged forms (Fig. 45(*f*)), using intersecting cable curves (like the bridge built in 1889 over the Tiber at Rome; also Mr. Lindenthal's 1910 design for the Quebec Bridge and his design in 1923 for the Sydney Harbor bridge); and the two-hinged type (Fig. 45(*g*)) with parabolic upper chord and polygonal bottom chord (like Mr. Lindenthal's 1899 design for the Quebec Bridge and his 1903 design for the Manhattan Bridge). In comparison with the last type, Mr. Moiseiff arrived at the same conclusion as the writers, namely, that the form now adopted for the Florianopolis Bridge (Fig. 45(*h*)) would be more advantageous, as it dispenses with the necessity of additional wind chords and presents a more pleasing appearance.

Mr. Moisseiff's contribution of an illustrated description of his design in 1907 for a proposed bridge of 1 200-ft. span over the Kill van Kull (Fig. 42), anticipating the Florianopolis design, helps to make the record complete.

Mr. Moisseiff would limit the economic applicability of the Florianopolis type to bridges where the live load is heavy and the span not too long. These suggesed limitations, however, appear to be subject to qualification, as is indicated by the recent adoption of the Florianopolis type in successive bidding competitions for two light highway bridges over the Ohio River. The writers are satisfied that the type can be applied advantageously for span lengths from 700 to 2 000 ft., as indicated by the two Ohio River spans of 700 ft., the Florianopolis span of 1 114 ft., Mr. Moisseiff's Kill van Kull design of 1 200 ft. span, his suggestion for the Manhattan Bridge design of 1 470-ft.

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2701.

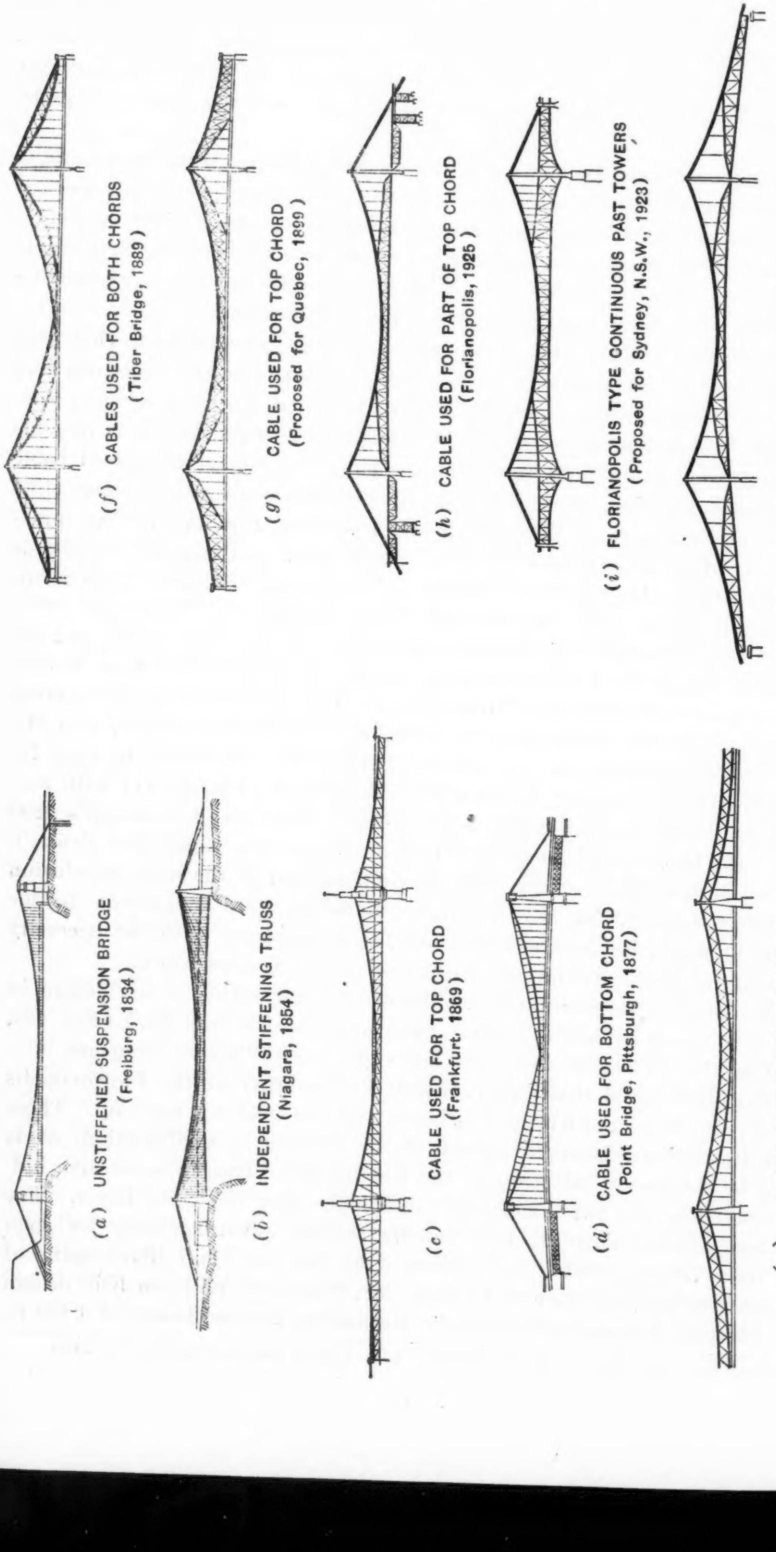

FIG. 45.—EVOLUTION OF SUSPENSION BRIDGE TYPES.

span, Messrs. Robinson and Steinman's design (1923) for the Sydney Harbor Bridge, with a main span of 1 600 ft., and a design study (1925) by the American Bridge Company for a railroad bridge of 2 000-ft. span. This range embraces the majority of suspension spans built, and future designs and competitions may further extend these limits of span length for the applicability of the Florianopolis type.

For the longer spans, Mr. Moisseiff advances the objection that the depth of the truss at the peak where the truss joins the cable becomes too deep. In the previously mentioned design by the American Bridge Company for a 2 000-ft. span, shown in Fig. 46, the truss depth at the peak was only 65 ft. The panels in this case were long, so that it was still possible to obtain a satisfactory slope for the diagonals. For spans still longer, it would probably be necessary to resort to a sub-divided panel arrangement to obtain a satisfactory diagonal slope. The writers agree with Mr. Moisseiff, however, that for extremely long spans, relatively shallow independent parallel-chord stiffening trusses would probably be more economical and satisfactory, unless the span is so long and the dead load relatively so high that no stiffening truss at all is necessary.

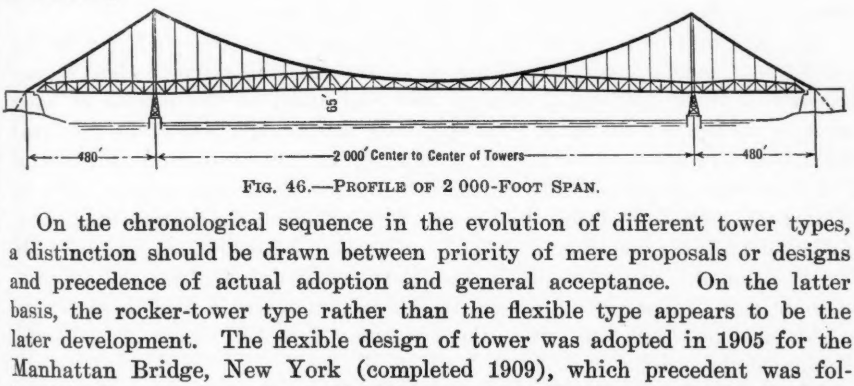

FIG. 46.—PROFILE OF 2 000-FOOT SPAN.

On the chronological sequence in the evolution of different tower types, a distinction should be drawn between priority of mere proposals or designs and precedence of actual adoption and general acceptance. On the latter basis, the rocker-tower type rather than the flexible type appears to be the later development. The flexible design of tower was adopted in 1905 for the Manhattan Bridge, New York (completed 1909), which precedent was followed in American practice in the Massena, Ohio, Bridge (1911), the Parkersburg, W. Va., Bridge (1916), the Kingston, N. Y., Bridge (1921), the Bear Mountain, New York, Bridge, (1924), and the Philadelphia, Pa., Bridge (1926). The rocker type of tower, first used in this country in the Muskingum River, Ohio, Bridge in 1915, did not receive general acceptance prior to its use in the Florianopolis Bridge (completed 1926); it has since been adopted for all new Ohio River bridges, including Portsmouth (1927), Point Pleasant (1928), Steubenville (1928), St. Mary's (1928), as well as for the three new Pittsburgh suspension bridges at Ninth Street (1926), Seventh Street (1927), and Sixth Street (1928). For two decades (1905-25) the flexible type of tower prevailed, almost without exception; then, commencing with the Florianopolis Bridge, the rocker tower design was generally accepted, with eight bridges of this type built in three years as against only three of the fixed base type undertaken in the same period. This succession of types in actual practice is what the writers had in mind in their remarks on the evolution of suspension-bridge tower design.

Mr. Moisseiff suggests that the choice between rocker and flexible types would depend on the height of the tower, since "a tall tower with a fixed base should hardly require more material than a rocker tower". There are other considerations, besides economy of material, that affect the choice, such as the difficulties and expense of cable erection adjustments with fixed base towers on the one hand, and the availability or lack of convenient means for holding the rocker towers during erection on the other. That the erection advantages of the rocker type are recognized by experienced erectors is indicated by the request of the American Bridge Company, during 1927, for permission to change the towers of the Mid-Hudson Bridge at Poughkeepsie, N. Y. (1 500-ft. span) from the flexible type to the rocker type under its contract.

The use of rocker towers greatly facilitates the necessary erection adjustments. Such towers are easily tilted back shoreward any amount as required for the connection of the back-stays to the saddle castings in bridges with eye-bar cables, or for the balancing of the catenaries during cable construction in bridges with wire cables. To hold the rocker towers in the required position during erection generally requires only a small amount of temporary material at the tower bases. With the use of fixed base towers, the necessary tower-top adjustments for cable erection are made more difficult.

Mr. Moisseiff cites the fact that the designers of the Detroit River Bridge (of 1 850-ft. span) have adopted towers of the fixed base type. Calculations recently made by Messrs. Robinson and Steinman, in the course of their erection studies for the cable contractors on this bridge, reveal the fact that the tower tops have to be moved shoreward the unprecedented amounts of 7.34 ft. on the American side and 5.04 ft. on the Canadian side, in order to balance the cable catenaries during erection. Pulling the towers back these indicated amounts would be a difficult and expensive operation, and methods for obviating this necessity also involve unanticipated difficulties and expense. If these results had been foreseen, it is doubtful whether fixed base towers would have been selected, since the adoption of rocker towers would have eliminated these difficulties.

The required tower movements during cable erection are most serious in designs with "straight" (unloaded) back-stays as in the Florianopolis, Bear Mountain, and Detroit Bridges. In such layouts, the rocker type of tower has especial advantages.

In addition to the advantages of economy of material in the towers and the elimination of expensive tower or saddle adjustments during erection, another advantage of rocker towers must not be overlooked; namely, the reduction of stresses (and consequent possible reduction of dimensions) in the masonry piers from the elimination of tower bending stresses.

Mr. Beanfield refers* to extensometer readings made on bridge members composed of groups of eye-bars and reports that he found considerable variation of stress in the individual eye-bars of a group. He attributes this to pin clearance, minor differences between pin-hole distances, and deformations in the eye-bar heads. The writers regret that no extensometer readings were taken on the eye-bar cables of the Florianopolis Bridge to ascertain, as sug-

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2705.

gested, if there is any great variation in stress in the individual eye-bars of each link. Neither, to their knowledge, have such measurements ever been made on the wire strands of large suspension bridge cables. In the case of a wire cable, the method of stringing (to equal sags) practically assures uniform tension of all the wires; and any local slackness of a wire is not material since it is only one of thousands of wires composing the cable and, therefore, represents only a very small fraction of 1% of the total section. Moreover, the compacting of a wire cable after stringing is such as to assure unified stress action; even if a wire is cut at any point, it will be found to have full stress again (transmitted by the frictional bond) a short distance away from the section.

Referring to the details of the top castings on the towers, Mr. Beanfield raises the question of secondary stresses arising from the frictional resistance to rotation of the eye-bars on the pins. In the case of the Florianopolis Bridge, the anticipated deflection movements under live load are so small that the resulting angle changes at the ends of the eye-bar cables would not produce any serious secondary stresses.

That the eye-bars rotated about the pins to take care of angle changes during erection was visibly demonstrated just after the first two panels of truss members at the continent end were erected by the "jinny-wink" standing on the 185-ft. span. When the jinny-wink was moved out to Panel Point 2 in order to erect truss members in Panels 2-4, there was a heavy local concentration on Point $C2$ of the eye-bar cable. This heavy load distorted the equilibrium polygon of the eye-bar cable so much—causing the eye-bars, $C0$-2 and $C2$-4, to rotate about Pin $C2$—that it was impossible to place the top chord, $C2$-4. The distance between Points 2 and 4 at the top chord was about 4 ft. too short for that chord to enter. The frictional resistance of the eye-bar stress on the pins was certainly overcome in this instance.

Mr. Beanfield, like Mr. Kuo, expresses concern on the security of the tower base castings against earthquake forces. Although provision against earthquake effects was not definitely specified or allowed for in the design of the Florianopolis Bridge, a few minutes with the slide-rule are sufficient to show that the tower base details are amply proportioned to resist displacement under the most severe seismic disturbances thus far recorded. The sixteen 3-in. dowels provide a total resisting section of 112 sq. in., or a safe shearing resistance (at 15 000 lb. per sq. in.) of 1 680 000 lb. The total weight of the suspension bridge (including towers, cables, back-stays, and suspended structure) is approximately 3 000 tons. This total weight would have to be subjected to a horizontal acceleration of 28% of gravity, or 9 ft. per sec. per sec., before the safe shearing resistance of the steel dowels would be exceeded. This acceleration (representing earthquake intensity) is seven times as high as that assumed in the design of the Carquinez Strait Bridge which, as cited by Mr. Beanfield, was designed with especial consideration to possible earthquake forces. Moreover, the foregoing comparison assumes the Florianopolis Bridge to be a free structure, with its entire inertia resisted solely at the tower bases. Actually, the structure has safe additional restraint in the fact that the tops of the towers are securely tied back to the massive concrete anchorages.

Mr. Beanfield voices a common misconception when he states that suspension bridges, in general, are not adaptable for locations subject to earthquake disturbances. Consideration of the functional characteristics of the suspension type should show that it is pre-eminently the bridge type best adapted to resist seismic forces. The longitudinal continuity of the principal element, the positive anchorage at each end, the comparative lightness of weight, the low center of gravity, and the characteristic resilience of the structure—all combine to give the highest security against earthquake effects. The contrary misconception is perhaps prompted by the thought of the relative flexibility of a suspension span; but this very flexibility gives safety against seismic shock or any suddenly applied force. A suspension bridge functions like an anchored steel spring, whereas other bridge types would act like stacks of blocks. Resilience is a valuable factor of safety against seismic effects. The Eiffel Tower would obviously be safer than Cleopatra's Needle in an earthquake; and the Florianopolis Bridge is likewise many times safer than the Carquinez Bridge against seismic effects. Where the suspension type is at all suitable, the writers would choose it every time in preference to a cantilever bridge in earthquake country.

On the question of the proportioning of the rocker bearings, the following notes may be of interest: For the base castings with a radius of contact of 12 ft., the designers adopted an allowable working pressure of 108 000 lb. per lin. in., equal to $750r$, or $375d$. This is five-eighths of the linear bearing pressure ($600d$) allowed by the specifications for steel rollers adopted by the Society's Special Committee on Specifications for Bridge Design and Construction.* Specifications of this form are based on theoretical derivations which take into account the anticipated elastic deformation of the line of contact into a narrow strip of contact area. They also take into account the supporting and relieving effect of the unloaded metal adjacent to the strip of contact. Actual tests on large steel roller or rocker bearings are needed in order to verify the theoretical deductions and to fix the coefficients more definitely.

Mr. Stowell remarks† that the element of cost is the "discordant note in the harmonious whole" of the Florianopolis Bridge project. This phase, however, may be viewed from a different angle. Had there been plenty of money available, a cantilever bridge of the type shown in Fig. 2(*c*)‡ would have been adopted. While this cantilever span would have held the record for monumental bridges in South America, still, no new features in bridge design and erection would have been introduced. Necessity is the mother of invention, and the scant supply of money taxed the ingenuity of the engineers connected with the development and execution of the project, and resulted in the production of the suspension type of structure with all its novel features.

Mr. Stowell expresses his conviction that the superiority of wire over eyebars for cables is so great that the question of relative cost should not be a controlling factor in deciding between them. On this point, there will be legiti-

* *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 1281.

† *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2707.

‡ *Loc. cit.*, May, 1927, Papers and Discussions, p. 710.

mate differences of opinion. Each type has its advocates and, apparently, a field for its appropriate utilization.

Mr. Stowell questions the reliability of the heat-treated eye-bars, on the score that the limitations of the testing machine necessitated the use of bars especially made for test instead of permitting selections from the finished lot to be used for testing. He cites the low elongation (3%) of one of the test bars as compared with the 7% average of the others, and he directs attention to the hazard of using eye-bars of different elongation in the same cable member. In justice to the eye-bars, however, it is only fair to note that the varying elongations shown in the test reports relate to the ultimate load and not to the working stresses below the elastic limit.

In further reference to the results of the full-sized tests on the eye-bars, the following explanatory notes are submitted: The American Bridge Company was under contractual obligation to have the test bars pass the minimum requirements of 75 000 lb. per sq. in. elastic limit, 105 000 lb. per sq. in. ultimate strength, and 5% elongation in 18 ft. As long as these requirements were met, the Company felt that here was an opportunity to experiment and ascertain whether eye-bars with 100 000 lb. per sq. in. elastic limit could be produced. This accounts for the variations in the elastic limit values, since the Company was not trying to produce uniform results. As a matter of fact, an elastic limit of 100 000 lb. per sq. in. was not quite attained, the nearest approach being 96 830 lb. per sq. in. While it is true that one bar failed in the elongation requirement, that same bar had an elastic limit of 82 780 lb. per sq. in., 10% higher than the guaranteed minimum, and an ultimate strength of 116 720 lb. per sq. in., 11% higher than the guaranteed minimum. On the additional bar which was tested, the elongation was 8.1%, or well above the guaranteed minimum.

Mr. Stowell appears to have misinterpreted the results of the eye-bar tests in his reasoning with respect to the variation of stress in the several eye-bars in any one panel. The entire theory of elastic structures is based on Hooke's law that, within the elastic limit of the material, unit stress is proportional to unit strain or distortion, the constant ratio between them being the modulus of elasticity. While it is true that the value of this modulus may vary slightly in different specimens of the same material, there is no such variation as would cause the distribution of stress among four eye-bars in one panel to be, as Mr. Stowell suggests, 35.2% in one bar and 21.6% in each of the other three bars. Before testing, each bar was marked with points 1 ft. apart for a distance of 13 ft. on two bars and from 34 to 38 ft. on the remaining eleven bars. After failure, the distances between these points were measured and the results were as plotted in Fig. 47. These curves show a very uniform elongation at all points on the bar except at the point of failure.

Mr. Stowell points out the exceptional reliability of wire for cables, in that two full-sized tests are made on each individual wire in addition to the automatic testing of quality and uniformity in the process of wire drawing. He also mentions other advantages of wire cables, including smaller percentage of details, elimination of points that are accessible to moisture and

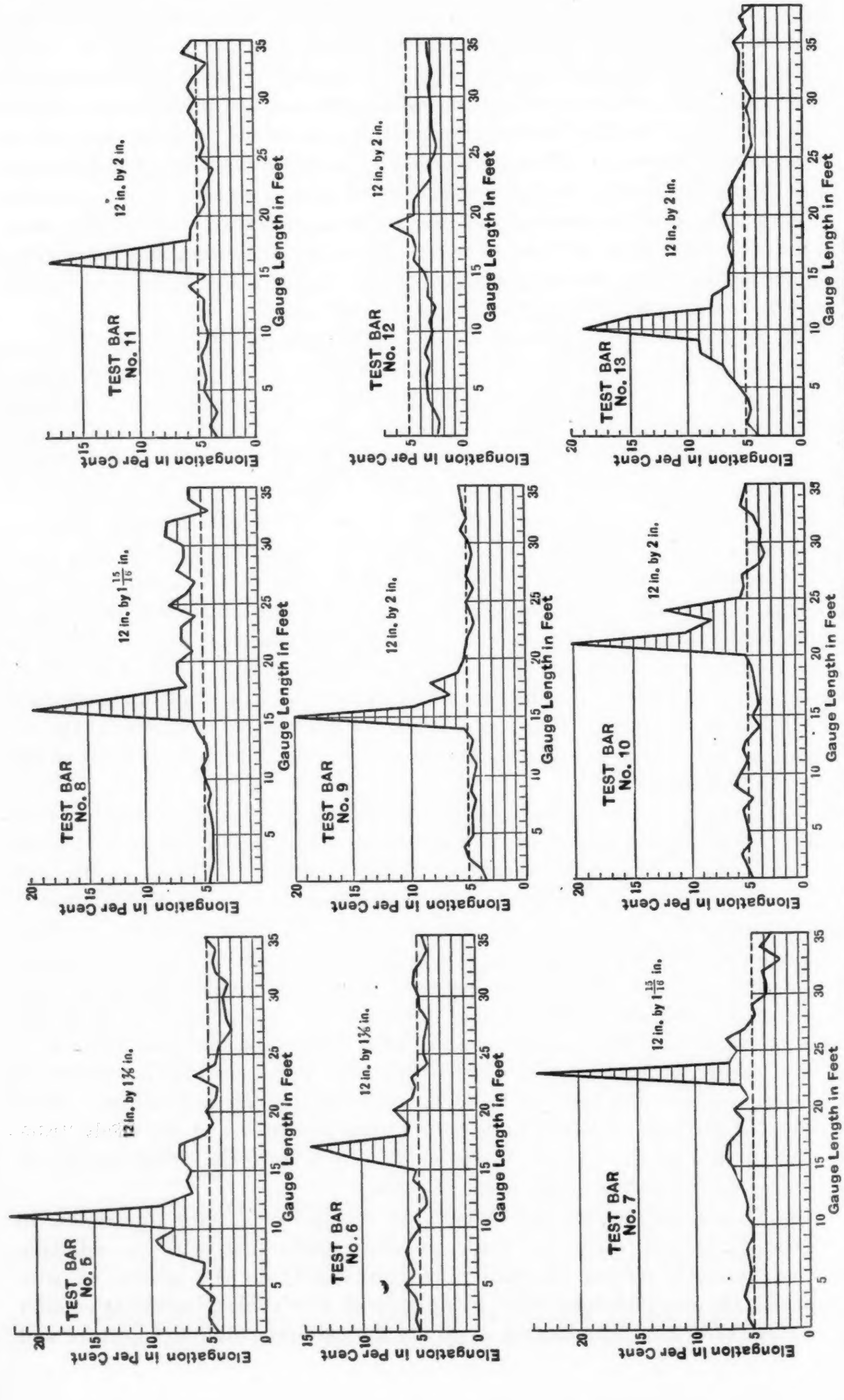

FIG. 47.—GRAPHIC REPRESENTATION OF ELONGATION OF EYE-BARS AT EACH FOOT OF GAUGE LENGTH.

difficult to paint, and the reduction of dead load by the much higher safe working stresses.

When Mr. Stowell comments on the extra material in pins and eye-bar heads for connections far outweighing the small sleeve nuts used for splicing wire, he overlooks the fact that the pins are used for attaching the suspenders in the case of eye-bar cables, while for the wire cable it is necessary to provide for this purpose cast-steel cable bands and high-tension bolts. Moreover, in the case of eye-bars, it is possible to vary the thickness of the bars in accordance with the stress variation along the length of the cable, instead of requiring the maximum section to be carried undiminished from tower to tower.

Mr. Stowell deprecates the policy of secrecy sometimes displayed by steel manufacturers, and he cites instances where the mystery was only pretense. His gratification at the abandonment of the element of secrecy in the manufacture of heat-treated eye-bars is shared by the writers.

Professor Cross confines his comments* to a discussion of the significance or lack of significance of relative deflections as a basis for the comparison of different bridge types. He indicates his agreement with the writers in doubting the propriety of applying to suspension bridges criteria for stiffness borrowed from other types of bridge construction. The writers are satisfied that larger deflections may properly be permitted in suspension designs than in other bridge forms. Such larger deflections are not a measure of impact effect. On the contrary, they may be regarded as a protection against impact. It is an established principle of mechanics that, if the same weight is dropped from equal heights on two beams of equal strength, the one with the greater deflection (in consequence of difference in form, material, or yielding supports) will suffer the smaller damage. The larger deflection reduces the kinetic impact effect.

A larger deflection of a suspension bridge, compared with that of a simple truss, does not mean overstress or greater wear. In other structures, deflection movements or vibrations may mean loosening of rivets, wear at joints, and reduction of useful life. In a suspension bridge, the principal carrying element (the cable) is free from joints in which rivets can loosen or wear can take place. The stiffening truss is of secondary importance. Moreover, it is so much shallower than the trusses of other bridge types, that a given deflection means much smaller stress and strain at the joints.

The objectionable psychological effect of vibration, as pointed out by Professor Cross, should be measured not by relative amplitude but by relative acceleration. The deflection movements of a suspension bridge are sluggish in comparison with the high-speed vibrations of a simple truss or an arch, or the jerky vibrations of a cantilever. Different bridge types obviously call for different criteria for permissible deflections, and for a different handling of the deflection problem in all its phases.

Concluding Remarks.—An interesting confirmation of the economy of material in the Florianopolis Bridge as compared with other suspension struc-

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 579.

tures is found in a paper by Mr. W. H. Thorpe on "Steel Bridge Weights".* Mr. Thorpe has analyzed the weights of steel per unit of live load for a large range of bridges and, after making corrections for differences in relative floor weight, has plotted the results as a function of span length to establish a mean curve for each respective type of bridge. In the graph of plotted points for suspension bridges, one point stands out prominently as the lowest one relative to the mean curve, and that is the point representing the steel weight factor of the Florianopolis Bridge. In comparison with the neighboring points in the plotted graph, the respective steel weight factors (per unit of live load) are:

Bridge.	Steel weight factor.
Elizabeth Bridge (951-ft. span, eye-bars)................	2.4
Florianopolis Bridge (1 114-ft. span, eye-bars)..........	1.6
Manhattan Bridge (1 470-ft. span, wire)................	2.5

The low weight factor recorded for the Florianopolis Bridge cannot be ascribed to the use of a timber floor, since correction for relative floor weight has been made in Mr. Thorpe's analysis; nor is the use of high-tension eye-bars a governing factor, since bridges with wire cables of higher unit tension are included in the comparison. The form of the structure, therefore, must be the principal reason for the low value of the weight factor. This is a gratifying independent verification of the steel-weight economy of the Florianopolis type of suspension structure.

Since this paper was written, the Florianopolis Bridge has been thrown open to highway traffic. A. Y. Sundstrom, M. Am. Soc. C. E., reports that the traffic amounts to approximately 1 000 vehicles and 10 000 pedestrians per day. The tolls charged are 100 reis (about 1 cent) per pedestrian and 1 milreis (about 10 cents) per vehicle, and thus amount to about $200 per day. A local firm, Corsini Brothers, has the contract, on a fixed annual basis, for the maintenance of the bridge and the collection of tolls, the surplus receipts being turned over to the Treasury of the State.

In conclusion, the writers wish to thank all those who have so generously enhanced the usefulness of the paper by the contribution of their discussions. If, as predicted in some of the discussions, the novel features in the design and construction of the Florianopolis Bridge will serve to enlarge the field of usefulness of suspension structures, the writers will feel fully repaid for their efforts.

Any departure from precedent in structural form, materials, or erection methods, is offered, not to replace, but to supplement those previously developed. The availability of alternative forms, materials, and methods should serve to increase the resources and to stimulate the resourcefulness of the bridge engineer. The healthiest condition in a profession is not the blind following of standardized precedent, but the open-minded evaluation and selection of the best of various possible solutions for a given problem.

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* *Engineering* (London), October 30, 1925.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE RELATION OF HIGHWAY TRANSPORTATION TO THE RAILWAY

Discussion*

By Ralph Budd, M. Am. Soc. C. E.†

Ralph Budd,‡ M. Am. Soc. C. E. (by letter).§—Mr. Crosby‖ properly questions the writer's "Inventory".¶ Values for highway right of way were not included and to that extent the figures given for investment in highway transportation were too low.

He also questions whether competition should not be permitted between different carriers even though such competition would add to the total cost of producing the transportation by the competing carriers. The writer does not believe in eliminating all competition, but, in the case of commercial highway transportation, the theory of regulated monopoly has much to commend it. Just how far this can go in practice is, of course, a question of judgment to be determined by the circumstances in each instance.

Mr. Gerry's plea** for quicker and cheaper transportation is in line with the ever-increasing need for efficiency. So far as passenger travel is concerned, the operation of through sleeping cars from the East to the West Coast, and *vice versa*, is practically all that the writer can think of that would eliminate delay and inconvenience in long-distance railway travel on the present-day fine, through trains. It is true that freight movement by rail can be expedited still more than it has been during the years since the World War, but most of the saving in time will come from avoiding terminal delays rather than from operating freight trains at higher speeds.

* Discussion of the paper by Ralph Budd, M. Am. Soc. C. E., continued from November, 1927, *Proceedings*.

† Author's closure.

‡ Pres., G. N. Ry., St. Paul, Minn.

§ Received by the Secretary, February 15, 1928.

‖ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 2043.

¶ *Loc. cit.*, May, 1927, Papers and Discussions, p. 793.

** *Loc. cit.*, November, 1927, Papers and Discussions, p. 2331.

By some further consolidation of railways, transfer of cars and the incident inspection will be unnecessary, and the cars may be kept moving more of the time.

The problem of taxation was discussed by Messrs. Hanna* and Hadley†. It seems a most difficult one to solve. The principles stated in the paper‡ still seem to be correct.

The obvious injustice of taxing the railways to provide highways for competitors should be removed, but not by imposing an unjust tax on other transportation agencies. The fairer method is to transfer the burden of taxation from the users of the railways to the users of the highways and to tax each of the latter on the basis of the proportion that its use bears to the total use made of the highways.

The trend of local passenger travel from the railways to the highways of the country, due to the increasing number of automobiles and highway motor buses and the improvement of roads, has continued uninterruptedly through 1926 and 1927. The decrease in railway passenger revenue in 1927 compared with 1925 was $78 800 000, and the decrease in the number of passengers handled was 51 000 000, or 5.8 per cent.

The astonishing thing is that many railway men thought and said in 1925 that the bottom had been reached in the decline of railway passenger travel. The conclusion seems inescapable that the railways have lost irretrievably the short-haul passenger business. The common sense course open to them is to substitute smaller, lower cost, gas or oil-electric units for steam-operated passenger trains, where trains are essential, and to eliminate unprofitable trains wherever they are not necessary. The latter process may be advanced by the operation of high-class, convenient motor-bus service on highways adjacent to and often parallel with the railways.

Probably the most striking statistics concerning the operation of the railways of the United States for recent years are those which show a decline in passenger revenues from $1 290 000 000 in 1920 to $980 000 000 in 1927, with practically no reduction in passenger train-miles during that period. The effect of this loss in passenger revenue is, therefore, a loss in net railway operating income of approximately $310 000 000 in 1927 compared with 1920, or the equivalent of 1% on $31 000 000 000. This is more than the total investment in the railways of the United States.

It is the 20 600 000 private automobiles, and not the 90 000 highway buses, that are responsible for this continuing decline in railway passenger revenue, although the total travel by bus is increasing, as is also the average length of ride per passenger. No one can say how much more railway travel will decline, but it is evident that the long-haul passenger business is going to remain with the railways just as the long-haul bulk freight will. To keep as much travel on railway trains as possible, the railways are justified in providing extra fine through trains and small economical units for short runs and branch-line operation. In many places highway buses can be

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1319.

† *Loc. cit.*, p. 1322.

‡ *Loc. cit.*, May, 1927, Papers and Discussions, p. 804.

substituted for unprofitable trains by co-operating with bus operators, or by the railways controlling the bus companies themselves.

Regulation of buses by the same commissions that regulate railway operation seems essential for complete correlation of rail and highway service. There is pending in Congress a bill to provide some Federal control of common carriers on highways and that bill, very wisely leaves most of such regulation to the State authorities. Beyond the issuance of certificates of public convenience and necessity authorizing interstate operation, there does not seem to be any real need for Federal jurisdiction.

During the last two years there has been substantial improvement in bus transportation. The capacity of buses has been enlarged, until the newest types carry thirty-eight passengers. Luxuries have been added in the way of air-cushioned seats, interior baggage racks, hot-water heaters, storm windows, and ventilators. Chassis also have been improved in riding qualities and motors of greater reliability and economy have been produced. The interiors of the latest buses are painted in pleasing colors giving a decorative effect comparable with the treatment of the fine transcontinental trains. Now, the use of aluminum alloy is proposed to lessen their weight by 3 000 lb., or more.

There is in active progress a series of consolidations of small bus companies into a lesser number of stronger and more efficient systems. In this respect, the bus situation, to-day, resembles the railway situation seventy-five years ago when amalgamation of many short lines into systems like the New York Central, New York, New Haven and Hartford, Pennsylvania, Baltimore and Ohio, and others was beginning. There is no doubt that larger and stronger bus companies will be able to improve upon the service of the small weak companies and to operate much more economically, just as the larger railway systems were able to improve upon the service of the many small railway lines operating independently. How far consolidation of bus lines will enable highway buses to compete more effectively with the railways for long-haul travel, is one of the most important transportation problems of the day.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

PRECISE WEIR MEASUREMENTS

Discussion*

By W. S. Pardoe, M. Am. Soc. C. E.

W. S. Pardoe,† M. Am. Soc. C. E. (by letter).‡—The authors have submitted the work of years to the profession in a concise and readable form. Having on several occasions worked with Professor Schoder, the writer believes his methods of measuring discharge up to 50 cu. ft. per sec. are equalled in accuracy by few laboratories and exceeded by none. His method of measuring the head, h, by means of a float and pointer reading to hundredths and interpolating thousandths of feet does not seem as accurate as the use of a hook-gauge with a vernier reading to thousandths and interpolating to ten thousandths.

Professor Schoder does well to call attention to the possible errors in the use of the weir in field work. An error of $2\frac{1}{2}\% \pm$, due to the condition of the crest as shown in Fig. 8§ between Series I and II (Dawson), is enough to give one pause. The enormous variation due to the height of weir and to the conditions of baffling and approach make the field results obtained by means of weirs more uncertain. If this method of measuring water, which is not a cheap or easy method, is used, all the circumstances surrounding the derivation of the proposed coefficients should be carefully examined and slavishly copied if any degree of accuracy is desired.

Professor Schoder's Formula (D)‖ for velocity of approach is a "jewel"; it lacks both rhyme and reason, but, as it is purely empirical, it should, like the pudding, be judged by the eating and not by its synthesis. Fig. 18¶ shows a remarkable accuracy for this formula.

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* Discussion of the paper by Ernest W. Schoder, M. Am. Soc. C. E., and the late Kenneth B. Turner, Esq., continued from March, 1928, *Proceedings*.

† Prof. of Hydr. Eng., Univ. of Pennsylvania, Philadelphia, Pa.

‡ Received by the Secretary, January 26, 1928.

§ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1422.

‖ *Loc. cit.*, p. 1434.

¶ *Loc. cit.*, p. 1437.

Francis derived his correction for velocity of approach rationally from the following expression for a rectangular orifice under a low head with an appreciable velocity of approach:

$$Q = 3.33\,L\left[\left(h_2 + \frac{v^2}{2\,g}\right)^{\frac{3}{2}} - \left(h_1 + \frac{v^2}{2\,g}\right)^{\frac{3}{2}}\right]$$

in which, h_1 and h_2 are the heads on the top and the bottom, respectively, of the orifice.

If h_1 is zero, as in the weir, this becomes:

$$Q = 3.33\,L\left[\left(h + \frac{v^2}{2\,g}\right)^{\frac{3}{2}} - \left(\frac{v^2}{2\,g}\right)^{\frac{3}{2}}\right]$$

v is the main velocity of approach, and the formula gives values much too low.

Professor Schoder cheerfully changes the minus sign to plus and then re-vamps the formula to take account of the mean velocity above and below the crest, putting it in the form of Equation (D):

$$Q = 3.33\,L\left(h + \frac{v_a^{\,2}}{2\,g}\right)^{\frac{3}{2}} + L\,h\,\frac{v_b^{\,2}}{2\,g}$$

This formula is not homogeneous; that is, it adds peaches to pears and gets apples. The term, 3.33, by derivation includes $\sqrt{2\ g}$, hence the quantity involving (v_a) is $\sqrt{\frac{\text{feet}}{t^2}} \times \text{feet} \times \text{feet}^{\frac{3}{2}}$, or cubic feet per second, as it should be; but the quantity involving (v_b) is feet $\times$ feet $\times$ feet, or cubic feet. However, in spite of this the formula gives excellent results and should be judged by them. Unfortunately, on account of the measurement of velocities in the approach canal Professor Schoder will find little or no other data to check his own, nor will the occasional user of the weir be inclined to add to his field work by such measurement; in fact, the standard weir, if such there is, is fast becoming a bit of laboratory apparatus.

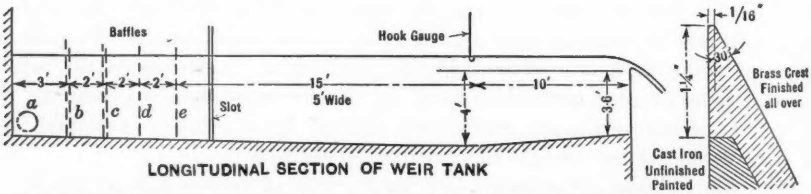

FIG. 56.—LONGITUDINAL SECTION OF WEIR TANK, CIVIL ENGINEERING DEPARTMENT, UNIVERSITY OF PENNSYLVANIA.

Herewith, in Figs. 56 to 61, inclusive, are shown coefficient curves of experiments made in the Hydraulic Laboratory of the Civil Engineering Department of the University of Pennsylvania. The discharges were measured in two 16 000-lb. weighing tanks, the time being found by using stop-watches calibrated by means of an electric chronograph. Fig. 56 shows a longitudinal section of the weir tank, 5 ft. wide. The water enters at Pipe (a), through drilled holes as indicated. Baffles (b) and (c) are 1 by 6-in. boards, 4 in.

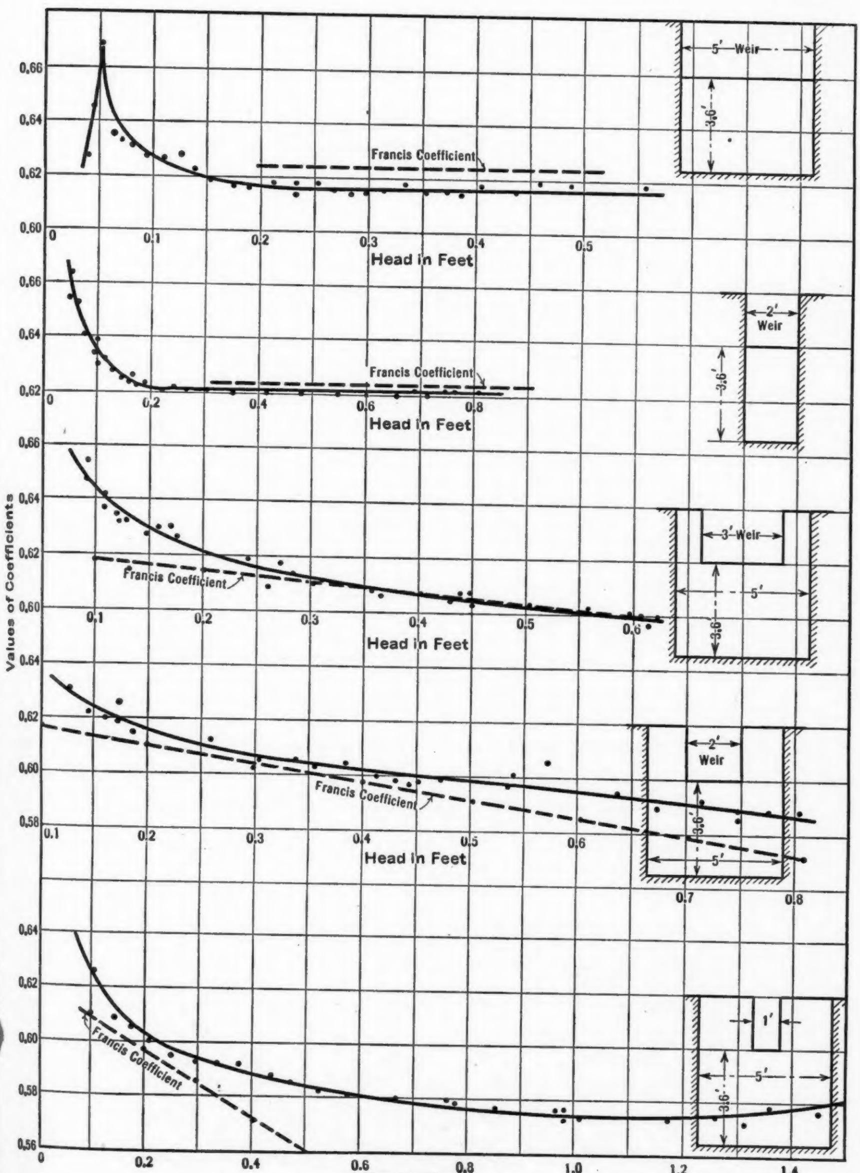

FIG. 57.—COEFFICIENTS OF RECTANGULAR WEIRS SUPPRESSED AND CONTRACTED.

apart vertically and 2 in. apart horizontally. Baffles (*b*) and (*c*) are cross-grids of 1 by 2-in. battens, 2 in. apart both horizontally and vertically. A cross-section of the crest of the weir is also shown.

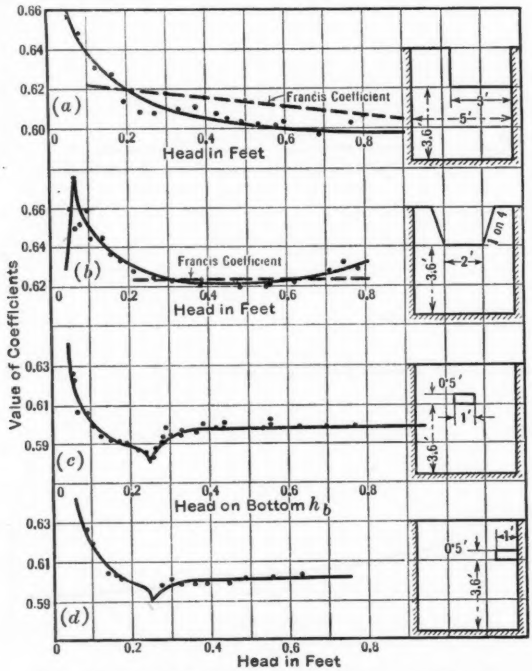

FIG. 58.—SECTIONS OF SEVERAL FORMS OF WEIRS.

In Fig. 57, showing coefficients of rectangular weirs suppressed and contracted, the crest in all cases is that given in Fig. 56. On these

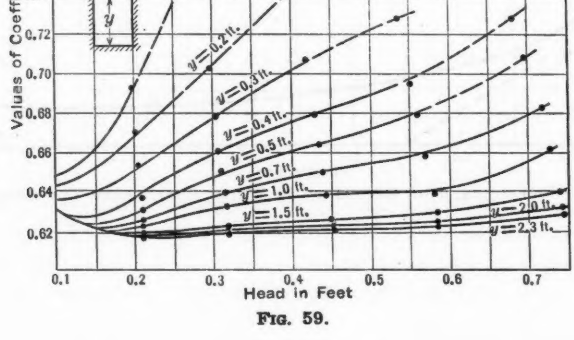

FIG. 59.

curves is shown also the Francis coefficient, 0.623, for suppressed weirs and $0.623\left(\frac{L - 0.02\,h}{L}\right)$ for contracted weirs. It seems to check best in the case of the 3-ft. contracted weir (Fig. 57 (*e*)). The formula for discharge used in connection with Fig. 57 is,

$$Q = C\,\frac{2}{3}\,b\,\sqrt{2\,g}\,h^{\frac{3}{2}}$$

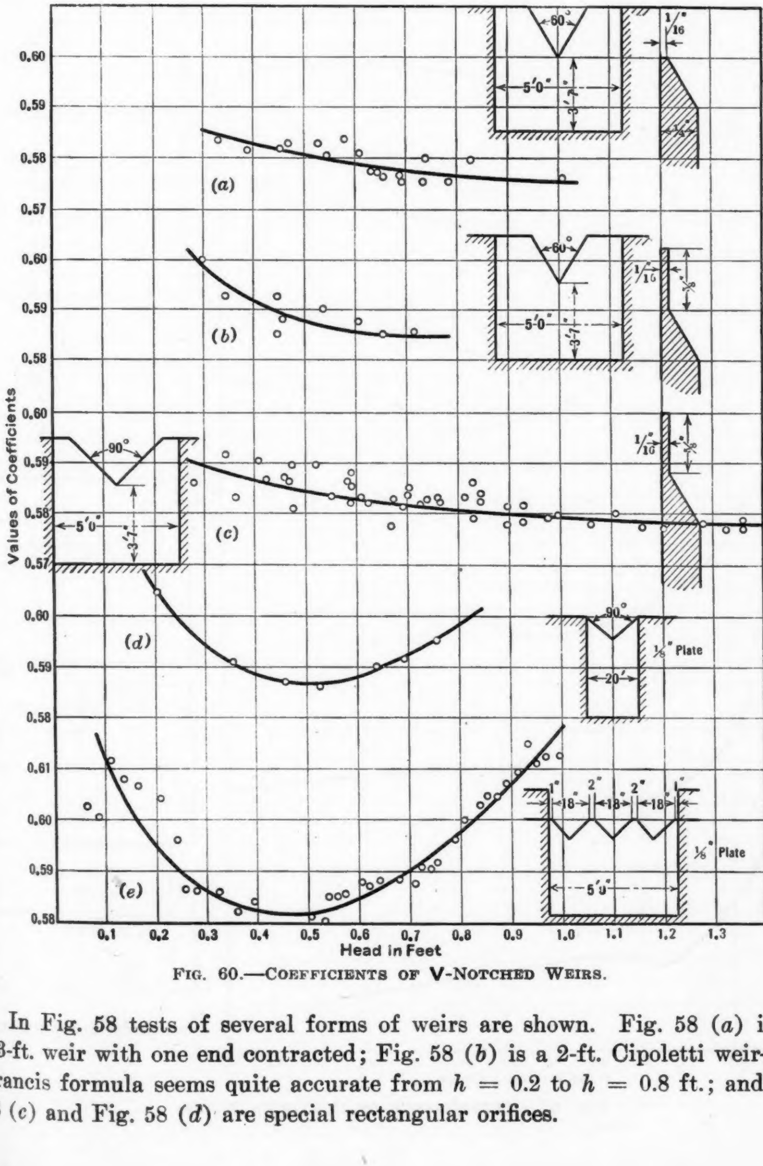

Fig. 60.—Coefficients of V-Notched Weirs.

In Fig. 58 tests of several forms of weirs are shown. Fig. 58 (*a*) is for a 3-ft. weir with one end contracted; Fig. 58 (*b*) is a 2-ft. Cipoletti weir—the Francis formula seems quite accurate from $h = 0.2$ to $h = 0.8$ ft.; and Fig. 58 (*c*) and Fig. 58 (*d*) are special rectangular orifices.

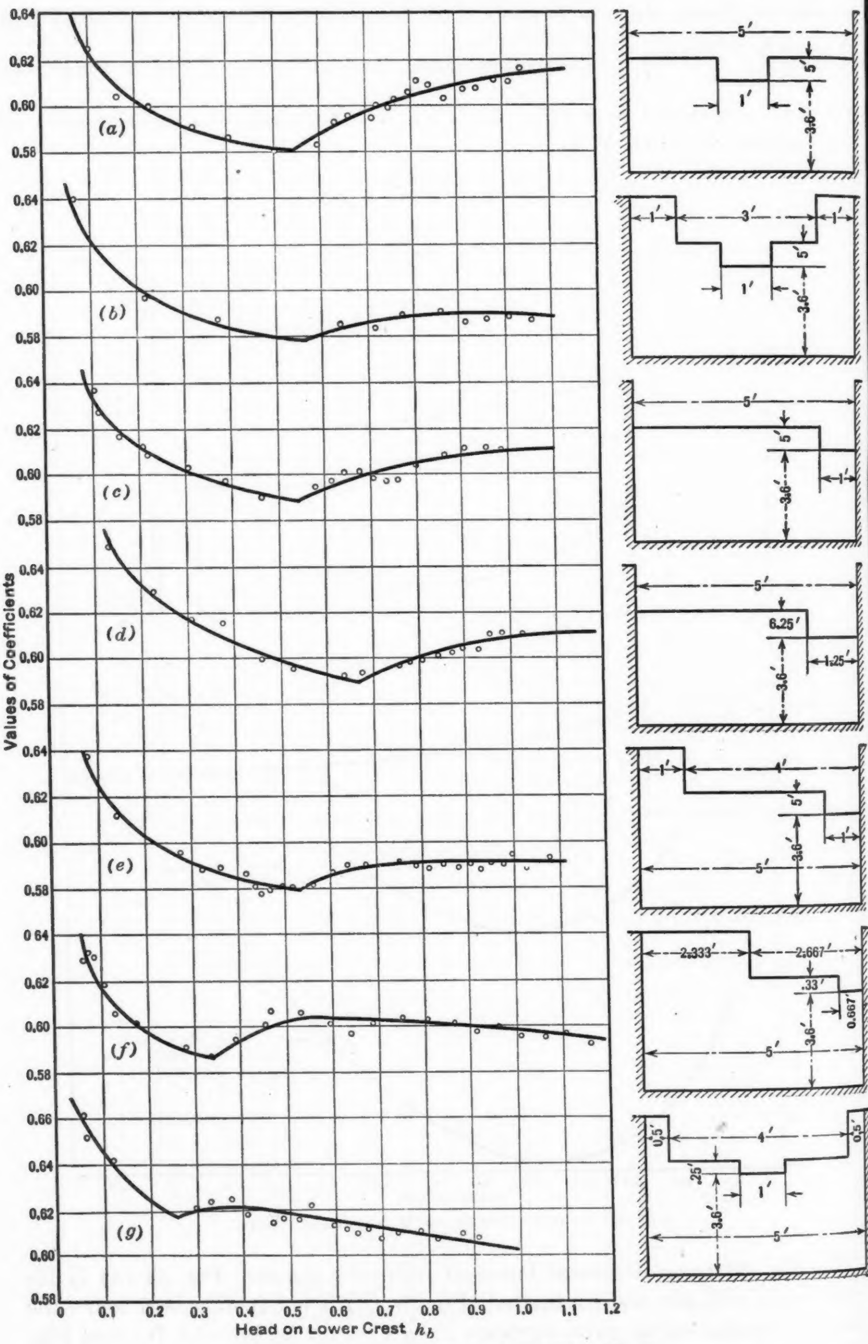

FIG. 61.—COEFFICIENTS OF COMPOUND WEIRS.

For Fig. 58 (c), the discharge formula is,

$$Q = C \frac{2}{3} b \sqrt{2g} \left(h_b^{\frac{3}{2}} - h_t^{\frac{3}{2}}\right)$$

in which, h_b and h_t are, respectively, the heads on the bottom and on the top of the orifice.

Fig. 59 is for a 2-ft. suppressed low weir with various heights of crest, y. Coefficients of V-notch weirs are given in Fig. 60. The formula is,

$$Q = C \frac{8}{15} \tan \frac{\theta}{2} \sqrt{2g}\, h^{\frac{5}{2}}$$

in which, θ is the angle of the notch. In each case the plate is painted steel. Fig. 60 (a) and Fig. 60 (b) show the effect of the form of the section of crest; Fig. 60 (d) and Fig. 60 (e), for a single and for a multiple 90° V-notch, respectively, give nearly the same results.

In Fig. 61 are given coefficients for a series of seven compound weirs. These coefficients can be closely approximated by assuming the weir to be made of two parts and taking coefficients from the preceding diagrams, using the formula,

$$Q = C \frac{2}{3} \sqrt{2g} \left(l\, h_b^{\frac{3}{2}} + L\, h_t^{\frac{3}{2}}\right)$$

in which, l and h_b refer to the length of crest and head, respectively, in the bottom notch; and L and h_t, similarly, on the top notch.

This work was done for the Water Supply Commission of the State of Pennsylvania under the direction and with the assistance of William Easby, Jr., M. Am. Soc. C. E. This type of weir is being used by the State for gauging streams, the low flows being confined to the small weir and the higher flows passing over the two crests.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

BASIC INFORMATION NEEDED FOR A REGIONAL PLAN

Discussion*

By Harold M. Lewis, M. Am. Soc. C. E.†

Harold M. Lewis,‡ M. Am. Soc. C. E. (by letter).§—The discussion on this paper indicates that regional planners are subject to two rather opposite faults—on the one hand to gather much more information than will prove useful, and on the other hand to obtain too little data in regard to existing topographic conditions. Somewhere between these two tendencies there must be a mean which will combine thoroughness with efficiency. Mr. Black‖ has expressed a disappointment that more information was not given concerning the types of basic information that will not prove essential for future regional planning surveys. The writer would not recommend that other regional plan organizations should spend as much time as was spent in New York on special research work in general and somewhat theoretical problems. He believes that the Committee on Regional Plan of New York and Its Environs was justified, for two reasons, for spending as much time and effort as it did. In the first place, while the Committee was working on a single region, it was financed by a foundation with National interests, the primary work of which is research; and, secondly, at the time when the studies were started, there was little published material available on such subjects. To-day, the condition is quite different. A large amount of information on the various standards of highway capacities, recreation use, population and housing densities, and community developments have been published within recent years. Much of it appears in the *Transactions* of the Society.

Mr. Leavitt¶ has referred to the "menace" of "the amassing of information that cannot be classified". It has been attempted to provide against

* Discussion of the paper by Harold M. Lewis, M. Am. Soc. C. E., continued from February, 1928, *Proceedings*.

† Author's closure.

‡ Executive Engr., Regional Plan of New York and Its Environs, New York, N. Y.

§ Received by the Secretary, February 16, 1928.

‖ *Proceedings*, Am. Soc. C. E., Papers and Discussions, November, 1927, p. 2371.

¶ *Loc. cit.*, December, 1927, Papers and Discussions, p. 2738.

this in the New York studies by publishing about nine survey volumes which will be a classified reference library of basic information relating to the New York Region. Based on this material there will be published two additional volumes dealing with the conclusions and proposals as they relate to the Regional Plan.

Several of the discussors have stressed the need for early official co-operation and for speed in arriving at specific conclusions. The writer believes that such procedure is highly advantageous and can be followed more and more in later regional plan activities. In the New York studies, which were started in 1921, it was necessary to "sell" the idea of regional planning in a district where State and municipal boundaries and rivalries made it exceedingly difficult. It would probably have been impossible to establish any degree of official co-operation at the start within a region which contained parts of three States and was subdivided into about 425 different political units. It was essential to get together a large amount of material before any attempt at such co-operation could be made.

To-day (1928), the need and justification of regional planning is fairly well recognized, and other regions can undertake similar activities with a considerable advantage over New York. This does not mean that they will not have to collect information under the various survey headings suggested, but that they should be able to make much better progress in compiling and making available such material.

It is important that basic survey information, particularly when gathered by a non-official organization, should be prepared for publication at the earliest possible date. In New York, this was done in the hope that when placed in the hands of local communities and planning bodies it would guide them indirectly toward a correct solution of their own problems. By deferring the publication of a comprehensive plan, it developed that many of the proposals which had been studied by the Regional Plan Staff were taken up by and received the local backing of different political units within the New York Region. It is believed that many excellent projects have been advanced toward consummation in this way, that might have been opposed if presented at an earlier date under Regional Plan auspices.

Basic information, such as collected for a regional plan, should prove useful for both regional and local purposes. It is impossible to draw any distinct line between information that is only essential for one scope of planning. The development of general principles of local application can be considered as regional planning. Their specific application would generally be local planning, although an example might well be given in a regional plan report to demonstrate their use. It is important, as Mr. Black has stated,* that the regional plan "should not concern itself with matters of purely local character".

Mr. Shurtleff's plea† for the reservation of open spaces adjacent to built-up areas is very timely. The writer believes that they are needed not only to "relieve monotony" and to prevent "depreciation in value", but also because

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2372.
† *Loc. cit.*, p. 2377.

intensive building cannot be carried out over large areas without causing an impossible traffic burden on street systems already fixed by existing developments, and creating excessive costs for the necessary systems of rail transportation. Buildings of large bulk, therefore, must be balanced against extremely low buildings or open spaces.

Mr. Eliot* has urged that the boundaries of a region be kept flexible. When the writer stated† that the "first item to be determined is the area to be studied", he did not mean to imply that its boundary should be inflexible, but only to state that before making a map of a region one must decide where he is to stop his map. The boundaries should be placed far enough out at the beginning to include all areas for which basic information will be required. As the work progresses more detail studies can probably be restricted to a smaller area.

The importance of the legal and financial surveys has been emphasized by Mr. Baker.‡ The Legal Division has been a very important part of the New York Regional Plan Staff, but most of its investigations come under the head of special studies and planning details, rather than basic information. The financial problem in a regional plan is more a matter of guiding expenditures along efficient lines than of finding new sources of revenue. It should be studied primarily with the aim of preventing the waste of public funds rather than of finding methods of increasing the normal capital expenditures.

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2731.
† *Loc. cit.*, September, 1927, Papers and Discussions, p. 1508.
‡ *Loc. cit.*, February, 1928, Papers and Discussions, p. 597.

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PAPERS AND DISCUSSIONS

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FORECAST: THE REGIONAL COMMUNITY OF THE FUTURE

Discussion*

By Thomas Adams,† Esq.

Thomas Adams,‡ Esq. (by letter).§—From certain points of view, the writer agrees with Mr. Kingery‖ that regional planning presents greater difficulties than city planning, but this should not be taken to mean that the technical preparation of a regional plan involves more labor and difficulty than the preparation of a city plan. The former covers a larger area than the latter, but it is not so hampered by the necessity of complying with legal requirements or of compromising with existing conditions. Subject to being based on sound principles and to being comprehensive in its treatment of problems, it may be said that the more elastic and the broader in outline the regional plan is, the better it will be.

Mr. Strong¶ shows by his emphasis on principles that he has the right conception of what is most needed. The regional plan is confined so much to problems of the future that planners need not be greatly influenced by political expediency or by pressure of vested interests in existing bad conditions. Therefore, they have no excuse for developing a plan that is not sound in principle, from the point of view of what is best for the community. As Mr. Fay states,** regional planning should be based on the policy of looking ahead over a long period of years, and in the development of its main features it should consider sociological as well as physical and economic problems.

It goes without saying that co-operation is the keynote of a regional plan. In fact, there can be no regional planning without much co-operation. The fact and extent of this co-operation between authorities of adjacent regions, which Mr. Kingery and Professor Lyle†† emphasize, are more important than the need of effecting governmental and administrative control

* Discussion of the paper by Thomas Adams, Esq., continued from January, 1928, *Proceedings*.

† Author's closure.

‡ Gen. Director of Plans and Surveys, Regional Plan of New York and Its Environs, New York, N. Y.

§ Received by the Secretary, February 17, 1928.

‖ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2417.

¶ *Loc. cit.*, p. 2419.

** *Loc. cit.*, p. 2422.

†† *Loc. cit.*, p. 2425.

as referred to by Mr. Knowles.* The writer predicts the operation of governmental control through the city organism rather than through the unorganized region. Nevertheless, Mr. Knowles has raised a most interesting point which will have greater significance in the future, namely, the desirability of new types of government for great urban regions. On this subject Professor Munro, of Harvard University, has recently written in support of those who advocate a regional form of government.†

Mr. Kingery's claim that some of the most obscure local officials are doing some very successful regional planning does not carry conviction. The work and projects of these officials are no doubt being developed in co-operation with his association and thereby are given a regional emphasis. It is most dangerous, however, to assume that the piecing together of different local projects is regional planning, even if this is done with co-operation. A regional plan should develop from considerations of the region as a whole down to the local considerations rather than take the form of a collection and composite of local projects.

There does not appear to be much ground for concern about lack of centralized organization or lack of capital, in spite of the truth of Colonel Wilgus' statement‡ that these represent two formidable obstacles to be overcome. They do present such obstacles in relation to all major engineering projects which are proposed within or without a regional plan, but they are no greater than have to be overcome in connection with all forms of planning and government. Since they have to be fought in any event, it is better that planners should have to fight them for some purpose that is really worth while. The writer's fear, in getting the dream of a better city brought into reality, is not for the lack of organization or capital, but for the lack of well-informed public opinion regarding what is true and false in the building of the city.

The differences between New York and other regional communities of the future, referred to by Mr. Leavitt,§ only relate to certain physical aspects. The New York Region contains the same social and economic elements and reflects the same tendencies of growth as other great urban aggregations.

No regional plan will correct blighted districts, congested areas, and the evils of skyscrapers. All it can accomplish is to show what is likely to happen from certain conditions and tendencies and to stimulate the operation of social and economic forces in those directions that will bring greater convenience, health, safety, and public well-being than is possible to be obtained with the present haphazard and disorganized growth within meaningless political boundaries.

The distinguished authorities who have discussed this presentation of a difficult subject, involving an attempt to make shrewd guesses into the future, have given the writer a greater confidence in the general predictions that he ventured to put forward with great temerity. If some of the things hoped for cannot be attained, there still remains the satisfaction of having hoped.

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 291.

† *The Forum*, January, 1928, p. 108.

‡ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2421.

§ *Loc. cit.*, December, 1927, Papers and Discussions, p. 2739.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

HISTORIC REVIEW OF THE DEVELOPMENT OF SANITARY ENGINEERING IN THE UNITED STATES DURING THE PAST ONE HUNDRED AND FIFTY YEARS

A SYMPOSIUM

Discussion*

By Messrs. Harry G. Payrow, George W. Fuller,† Harrison P. Eddy,† and George Truman Palmer.†

Harry G. Payrow,‡ M. Am. Soc. C. E. (by letter).§—As a resident of Bethlehem, Pa., the writer was interested in Mr. Fuller's reference‖ to the first water-works pumping plant in the United States, which was operated there. A few years ago he saw the original plans of both the pumping station and a layout of the distributing system. Both plans were well executed, the lettering being in German script. Fig. 3 is a traced copy of the distributing system that may be of some interest, at least as to its comprehensiveness.

The lignum-vitæ pump first successfully forced water from a spring up to a wooden tank on May 27, 1755, and on June 27 of that year, the regular operation of the Bethlehem Water-Works was commenced (see Fig. 4). The pump lift was about 45 ft. The pipes were generally bored logs of hemlock, although some that were recently excavated were gum and walnut. Fig. 5 shows a bored hemlock water pipe, laid about 1754, which was excavated in 1927. Apparently, a considerable length of lead pipe was used for connections to cisterns and buildings. As the pipe was laid, offsets were taken, chiefly to

* Discussion of the Symposium on Historic Review of the Development of Sanitary Engineering in the United States During the Past One Hundred and Fifty Years, continued from February, 1928, *Proceedings*.

† Authors' closures.

‡ Asst. Prof., Civ. Eng. Dept., Lehigh Univ., Bethlehem, Pa.

§ Received by the Secretary, December 19, 1927.

‖ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1594.

buildings, thus showing considerable foresight on the part of the builders in locating the water mains permanently.

It might be further stated that these early Moravian settlers fully appreciated the value of protecting their famous spring from pollution. As early as 1747 some concern was expressed as to the purity of Bethlehem's water supply. The Village Board ordered the spring to be enclosed with a fence,

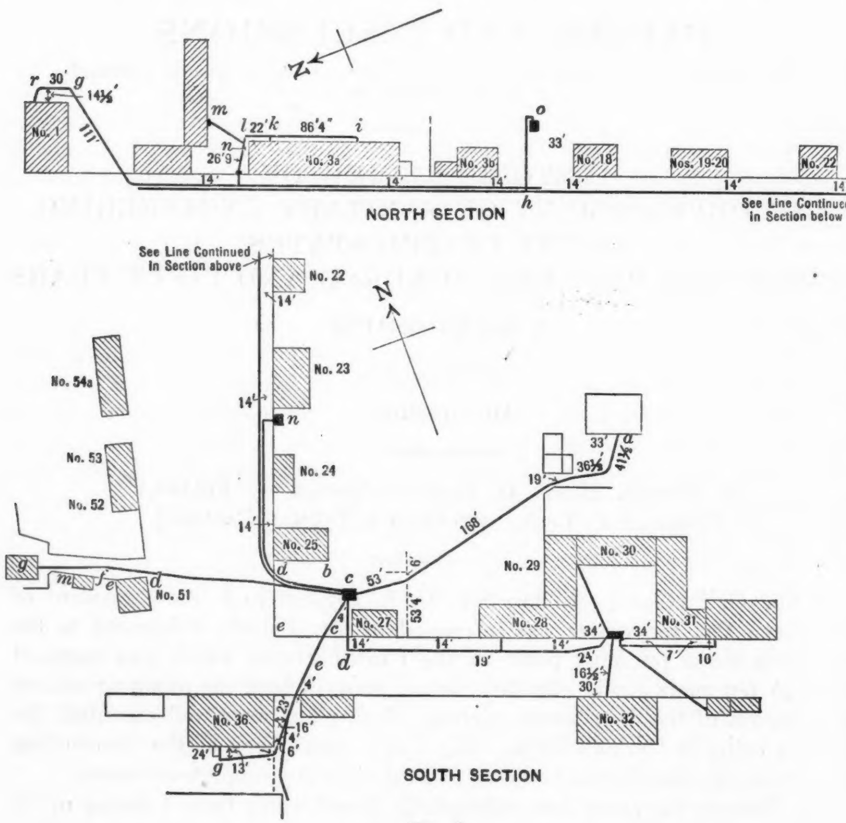

FIG. 3.

and it was to be cleaned "in the light of the moon". Later, it was ordered that only those who understood it should attempt to clean the spring. This may have referred to the folk notion that the state of the moon must be heeded.

GEORGE W. FULLER,* M. AM. SOC. C. E. (by letter).†—The writer is appreciative of the several discussions amplifying the paper, which at best could only be brief and fragmentary on a subject with so many ramifications. As Mr. Fenkell indicates,‡ it is to be regretted that recent performances of steam

* Cons. Engr. (Fuller & McClintock), New York, N. Y.

† Received by the Secretary February 25, 1928.

‡ *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 606.

FIG. 4.—PUMP-HOUSE AS IT LOOKS TO-DAY.

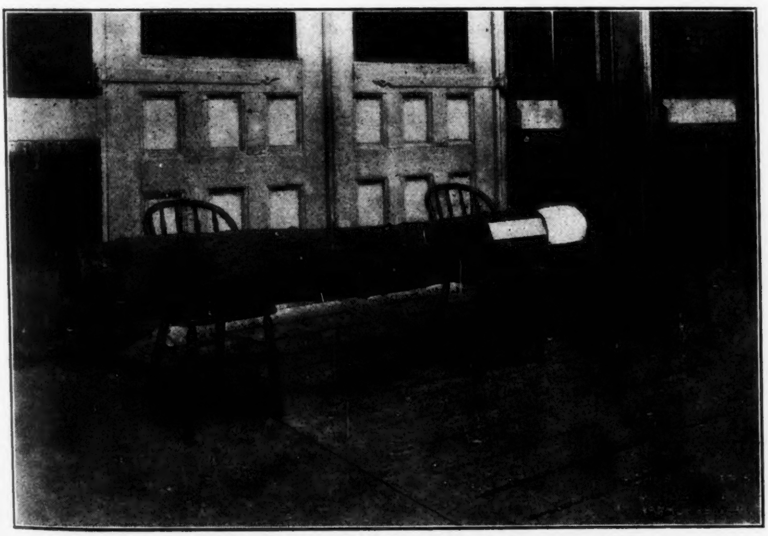

FIG. 5.—BORED HEMLOCK WATER PIPE, LAID ABOUT 1754.

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turbines and electric motor-driven centrifugal pumps, as well as data on maximum steam pressures used, were not included in the paper. However, it is probably best now to deal with them by reference rather than to include them in this discussion. These and a number of statistical and other data of interest will be found in the Manual of Water Works Practice issued by the American Water Works Association in 1925.

Harrison P. Eddy,* M. Am. Soc. C. E. (by letter).†—It is always difficult to be sure of the accuracy and completeness of historical data. In presenting his paper, the writer felt that, while the information given therein was based on good authority, much other material of interest was preserved in local records or in the memories of a few. For this reason, it is gratifying that Mr. Allen‡ and Mr. Webster§ have added interesting data relative to the early history of sewerage in this country, particularly as regards New York, N. Y., and Philadelphia, Pa.

The writer is inclined to believe that Mr. Kiersted has put it a little too strongly when he refers‖ to "the insurmountable difficulties of attempting to combine, successfully, the purely mechanical process of sedimentation with the biological process of sludge digestion" in the Imhoff tank. The successful operation of Imhoff tank installations indicates that the difficulties are not insurmountable, and it is likely that Imhoff tanks will continue to occupy an important place among the devices for the treatment of sewage. The writer agrees with Mr. Kiersted in the matter of adequate preliminary sedimentation with the view of removing as large a proportion of suspended solids as possible, prior to subsequent treatment. The trickling filter functions excellently with well clarified tank effluent, although the removal of colloidal solids will materially increase its capacity, particularly with strong sewage. The periodic unloading of trickling filters is an established phenomenon, but the material unloaded by no means consists of the solids applied. Much of it is composed of organisms having their origin in the filter and would occur irrespective of the degree of removal of colloids. Sludge disposal is still a problem of importance and is often difficult to accomplish satisfactorily. As Mr. Kiersted notes, lagooning of sludge at isolated sites is often both economical and satisfactory; in other cases, however, the problem cannot be solved so easily.

Professor Gregory's emphasis of the advantages of the separate system of sewers will appeal to many engineers. Under certain conditions separate sewers are advisable and frequently decision between the two systems is difficult and is dictated by local conditions rather than by general principles. In the case of large streams, storm-water overflows may not cause serious pollution; but, as Professor Gregory indicates,* such overflows may cause decidedly objectionable conditions in small streams. The common misuse of separate sewers and storm-water drains is a reflection on American municipal conditions.

* Cons. Engr. (Metcalf & Eddy), Boston, Mass.

† Received by the Secretary, February 27, 1928.

‡ *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2756.

§ *Loc. cit.*, January, 1928, Papers and Discussions, p. 303.

‖ *Loc. cit.*, December, 1927, Papers and Discussions, p. 2753.

Dr. Imhoff's references* to sludge digestion are of interest, although data from the Imhoff tanks at Schenectady, N. Y., indicate slightly higher temperatures in the sludge compartments than in the sedimentation compartments. This fact suggests thermolytic action as a result of the digestion processes. Confirmatory evidence from other installations is desirable in this connection.

The several discussions emphasize two points: First, the great advance made in sewerage and drainage since the middle of the Nineteenth Century; and, second, the fact that much remains to be established in the scientific principles underlying proper sewage treatment and disposal.

DR. GEORGE TRUMAN PALMER† (by letter).‡—In closing the discussion on ventilation, the writer would like to call attention to certain mitigating circumstances which have tended to retard ventilation progress.

People can exercise choice in what they drink, what they eat, and what they wear; but, in the matter of air, they are forced to be communistic. At least, the occupants of the same enclosure are forced to share the same air environment. Within the same room, one person cannot have low temperature and another a higher temperature at the same time. So long as "some like it hot and some like it cold" there will continue to be differences of opinion on air comfort, and, in consequence, a certain amount of dissatisfaction. In the effort to eliminate the objectionable and the unhealthful in ventilation, sanitary experts can never hope to surmount entirely this question of personal preference and individual idiosyncracies.

The writer's criticism of certain existing legal requirements as to ventilation are made on the ground that they hinder progress. The 30-cu. ft. requirement obstructs the development of new methods. Window ventilation with gravity exhaust has its application. There is scientific evidence and practical experience to support this method. Many State laws, however, prohibit this practice. It is fully believed that there is less evidence to justify these laws than there is to sustain them.

If the laws were altered to permit real competitive development of ventilation methods, the better methods would eventually reveal themselves by their survival. The substitution of the 15-cu. ft. requirement in place of 30-cu. ft., would make this competition possible and the experimental evidence is surely sufficient to provide assurance that health would not suffer by the change.

Mr. Samuel Lewis§ is quite right in criticizing the writer's statement about the desirability of reducing air-flow requirements as not representing the consensus of opinion. This should have been stated more carefully. The writer feels, however, that even if a majority of ventilating engineers may not agree with this attitude, nevertheless such a change is warranted by the experimental evidence available and by practical experience.

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2755.

† Director of Research, Am. Child Health Assoc., New York, N. Y.

‡ Received by the Secretary, March 1, 1928.

§ *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2770.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

NEW THEORY FOR THE CENTRIFUGAL PUMP

Discussion*

By Messrs. Mario Medici, A. H. Blaisdell, and Morrough P. O'Brien.

Mario Medici,† Esq. (by letter).‡—The author's statement as to the fallacies of modern theories of centrifugal pumps is well founded only as applied to theories that are typical of British and American texts, or of the old German method (Neuman, for instance). On the Continent, such methods, which are all based on the Eulerian theory, are being displaced by new ideas developed from the deductions of modern hydrodynamics. This is especially true of calculations for the design of centrifugal pumps, turbo-blowers, and compressors.

Professor A. Stodola§ and the writer‖ both give (see Fig. 19) the theoretical value of H as $\frac{v_2^2}{2\,g}$, or $\frac{u_2^2}{2\,g}$, when the discharge is zero. This seems reasonable when considered in the light of the more exact characteristic equation developed by the writer for the theoretical potentiality of centrifugal impellers. However, if the discharge valve is opened a little, that is, if the discharge is almost zero, there must obviously be a certain difference between the actual and theoretical head of the impeller and the pump. This difference is the only thing that can explain the actual quantity-head curves, such as A (Fig. 19). How does the author make his theory harmonize with characteristic curves of this kind?

The writer does not concur in the opinion of Professor Sherzer as to the effect of the vane angle, β_2, on the shape of the pump characteristic. For an impeller without casing, the author accepts, as being substantially correct,

* Discussion of the paper by A. F. Sherzer, Assoc. M. Am. Soc. C. E., continued from March, 1928, *Proceedings*.

† Prof. of Mech. Eng., Polytechnic School of Naples, Naples, Italy.

‡ Received by the Secretary, December 5, 1927.

§ "Steam Turbines," VI Edition, p. 1045, by A. Stodola, Springer, Berlin, Germany.

‖ "Contributo alla teoria delle turbomacchine centrifughe," by Mario Medici, *L'Industria*, September, 1926.

the results usually given in textbooks. The writer has shown§ that at supplementary angles of β_2 ($\beta_2 = 60°$ and $\beta_2 = 120°$, for example), the same trial characteristics occur and that the best values of β_2, for centrifugal impellers, are those between 24 and 30 degrees. This is in perfect harmony with current practice. Tests have verified the conclusion that both the vane angles, β_1 and β_2, and the number of impeller vanes, z, control the shape of the quantity-head curve.

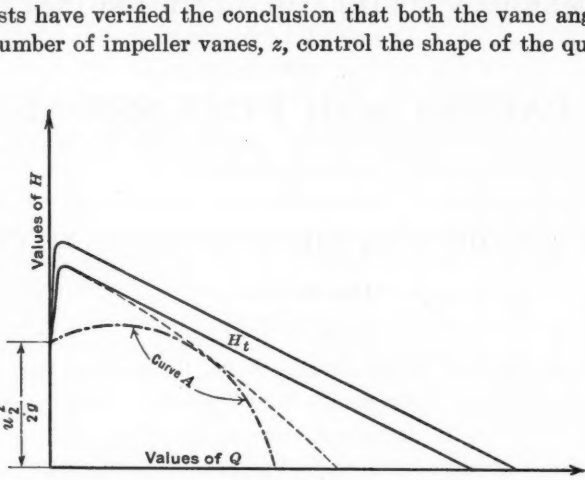

FIG. 19.

On the other hand, Professor Sherzer is to be congratulated on his conclusions as to the effect of casings and on his valuable suggestion to mount the impeller in a plain circular casing with constant cross-section. Working independently, and analyzing critically the real effect of the casing on the performance of centrifugal pumps, the writer reached the same conclusion but only with regard to series pumps (1927). The results have been published in the Italian technical press* after a delay due to a desire to obtain some proof of an experimental nature. The writer has not yet completed this experimental verification and, therefore, he is especially glad to learn of Professor Sherzer's test results as they apply to circular case centrifugal pumps. Some of these results, as, for instance, the extreme flatness of the efficiency curves, are in perfect harmony with the writer's theoretical deductions.

However, Fig. 16† is not clear. It seems to contradict the theory given by the author. According to his theory, the head at zero discharge must be, $H_2 = \frac{v_2^2}{2g}$, or $\frac{v_3^2}{2g}$, if the effect of the side-walls is ignored. Referring to Fig. 16, if the impellers are both of the same diameter and rotate at the same speed, how can the value of the head at zero discharge in a circular casing be so different from that in a volute casing? What does Professor Sherzer mean to illustrate with the values, 284 ft. and 342 ft., as the total head at zero discharge, using volute and circular casing, respectively?

The writer hopes that these comments will not be considered as destructive criticism, but as an effort to become better acquainted with the results of this

* *L'Industria*, December, 1927.

† *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1798.

very interesting investigation. In further support of these views, the writer submits an abridgment of his paper, "The Theoretical Potentiality of Centrifugal Impellers",* translated from the Italian and paraphrased as follows.

Designers of centrifugal pumps, fans, turbo-blowers, and compressors are recognizing, more and more, the need of abandoning the ancient Eulerian theory in favor of a more complete one. Calculations made by the old theory not only give results that are always greater than the actual values, but also (especially within forward and radial vanes) results that are far from being explained by the customary shape and size of the actual characteristic curves and the values of the impeller efficiency.

The Eulerian theory takes no account of the number of impeller vanes, z, in determining theoretical potentiality. It is based on conditions that obtain at the middle of the fluid stream passing through the impeller and diffuser vanes. In this theory the fluid trajectories and the vanes are considered congruent. With such an assumption it is easy to compute the theoretical change of energy in the fluid through the runner. This computed value will never be the same as that found under actual working conditions because the number of vanes, z, is finite, and the Eulerian theory tacitly assumes that z is infinite. In practice, it is only very close to the impeller vane that the fluid really travels in a direction that can be controlled by the inclination of the vanes. In the space between two adjacent vanes, variations both as to velocity and direction of flow appear in the stream (Fig. 20). Both these influences are responsible for reducing the actual head to less than the theoretical result as determined by considering the number of vanes infinite.

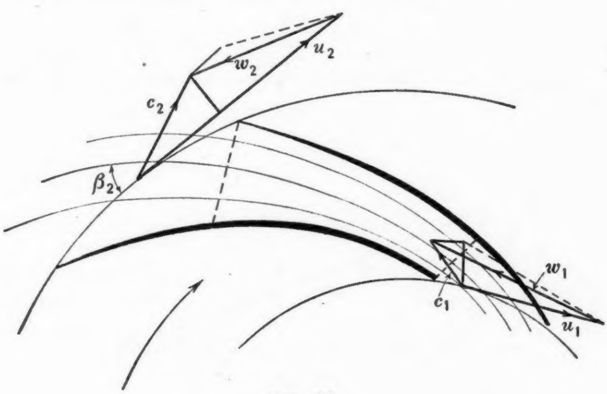

FIG. 20.

It is the variation in velocity (see Fig. 20) between vanes that forms a basis for the conception of "vane pressure"; that is, difference in pressure must undoubtedly take place between two centrifugal impeller vanes in order that power be transmitted. The effect of variations in the direction of flow is to make necessary an exaggeration of the vane angles as would be determined by the Eulerian theory.

* "Contributo alla teoria delle turbomacchine centrifughe," by Mario Medici, *L'Industria*, September, 1926.

A precise determination of the effect produced by each of the principal vane parameters (z, the number of vanes; β_{1c}, the vane angle at the inlet; and β_{2c}, the vane angle at the outlet), on the theoretical characteristics of the impeller is possible only by studying the fluid stream along the impeller; that is, by applying the methods of modern hydrodynamics, which in some respects has been as slow in making progress as aerodynamics. Under the present state of applied hydrodynamics, it seems to be much more rational to elaborate the ancient theory again, making proper allowances for β_{1c}, β_{2c}, and z, on the theoretical potentiality of centrifugal runners.

Usually, the theoretical potentiality of centrifugal impellers is expressed as:

$$H_t = \frac{u_2\, c_{2u}\; u_1\, c_{1u}}{g} \text{.......................} \quad (18)$$

in which,

H_t = theoretical head at the impeller;

g = acceleration due to gravity;

u_2 = peripheral speed of the impeller at the outlet;

u_1 = peripheral speed of the impeller at the inlet;

c_{2u} = the peripheral component of the absolute outlet velocity of the fluid, reduced because of "vane pressure"; and

c_{1u} = the peripheral component of the absolute inlet velocity of the fluid, reduced because of "vane pressure".

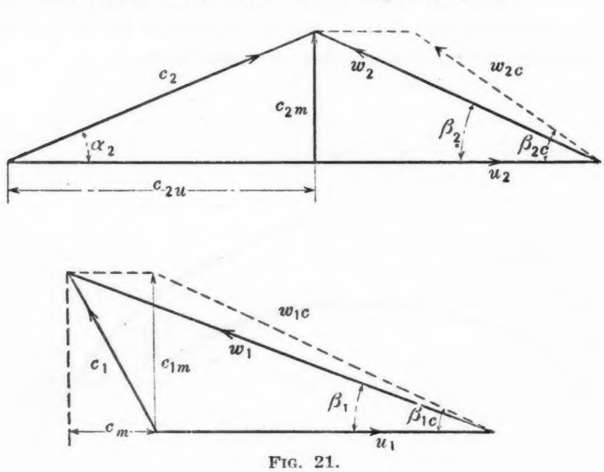

FIG. 21.

From the speed triangles (Fig. 21):

$$c_{2u} = u_2 - \sqrt{W_2^{\,2} - c_{2m}^{\,2}} = u_2 - \sqrt{W_2^{\,2} - \frac{Q^2}{\pi^2\, D_2^{\,2}\, b_2^{\,2}}} \text{.......} \quad (19)$$

$$c_{1u} = u_1 - \sqrt{W_1^{\,2} - c_{1m}^{\,2}} = u_1 - \sqrt{W_1^{\,2} - \frac{Q^2}{\pi^2\, D_1^{\,2}\, b_1^{\,2}}} \text{.......} \quad (20)$$

so that,

$$H_t = \frac{u_2^{\,2} - u_1^{\,2}}{g} - \frac{u_2}{g}\sqrt{W_2^{\,2} - \frac{Q^2}{\pi^2\, D_2^{\,2}\, b_2^{\,2}}} + \frac{u_1}{g}\sqrt{W_1^{\,2} - \frac{Q}{\pi^2\, D_1^{\,2}\, b_1^{\,2}}} \text{..} \quad (21)$$

in which,

Q = the delivery or capacity of the impeller;
D_2 = the outlet diameter of the impeller;
b_2 = the outlet width of the impeller;
D_1 = the inlet diameter of the impeller; and
b_1 = the inlet width of the impeller.

On every cylindrical element between D_2 and D_1:

$$d\,p = \frac{1}{2\,g}\,(W_f^2 - W_b^2) \qquad (22)$$

in which, $d\,p$ equals the difference in pressure between the two sides of a vane, and W_f and W_b are the relative fluid velocities on the front and on the back of the vane on the aforementioned element of the impeller.

According to the hypothesis of Kucharski* the relative fluid velocities on the front and on the back of the vane are quite different, but along the arcs, $A\,C$ and $E\,F$ (Fig. 22), the velocities may be considered as uniform and equal, respectively, to W_1 and W_2. At the point, D, on the back of the vane the relative fluid velocity caused by "vane pressure" will be W_{1b} which is greater than W_1. Similarly, at Point G_f for the same reason, the relative fluid velocity will be W_{2f} which is less than W_2. At Point D_f, on the front side of the vane the relative fluid velocity will be W_{1c} which is less than W_1, and at Point G_b, the relative fluid velocity will be W_{2b} which is greater than W_{2c} (Fig. 22). From Equation (22), then:

$$d\,p_G = \tau_2\,d\,p = \frac{1}{2\,g}\,(W_{2b}^2 - W_{2f}^2) = \frac{1}{2\,g}\,(W_2^2 - W_{2f}^2) \qquad (23a)$$

$$d\,p_D = \tau_1\,d\,p = \frac{1}{2\,g}\,(W_{1b}^2 - W_{1c}^2) \qquad (23b)$$

in which, τ_1 and τ_2 are two coefficients to allow for the difference in the distribution of the "vane pressure". They generally lie between 1.2 and 0.8. If p_m is the average value of "vane pressure":

$$d\,p = \frac{p_m}{\zeta} \qquad (24)$$

in which, ζ is the specific weight of the fluid.

The peripherical component of the "average vane pressure", $p_m \sin\beta$, gives on a surface element of the vane:

$$d\,s = d\,b\,\frac{d\,r}{\sin\beta} \qquad (25)$$

the turning moment:

$$d\,m = p_m \sin\beta\; r\,d\,s - p_m\,r\,d\,r\,d\,b \qquad (26)$$

Assuming that p_m is uniformly distributed over the central portion of the vane, $D_f\,G_f$ (Fig. 22):

$$m = p_m \int_{r_D}^{r_G}\int_0^{b_m} r\,d\,r\,d\,b = p_m\,b_m \int_{r_D}^{r_G} r\,d\,r \qquad (27)$$

* Kucharski, "Strömungen im rotierenden Kanal," *Zeitschrift für das Gesamte Turbinenwesen*, 1917, s. 201.

Since:

$$\int_{r_D}^{r_G} r\,d\,r = \varepsilon \int_{r_1}^{r_2} r\,d\,r = \varepsilon \left(\frac{D_2^{\,2} - D_1^{\,2}}{8}\right) \ldots\ldots\ldots\ldots (28)$$

in which, ε is a coefficient, defining the shape of the vane, then:

$$m = \varepsilon\, p_m\, b_m \frac{(D_2^{\,2} - D_1^{\,2})}{8} \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

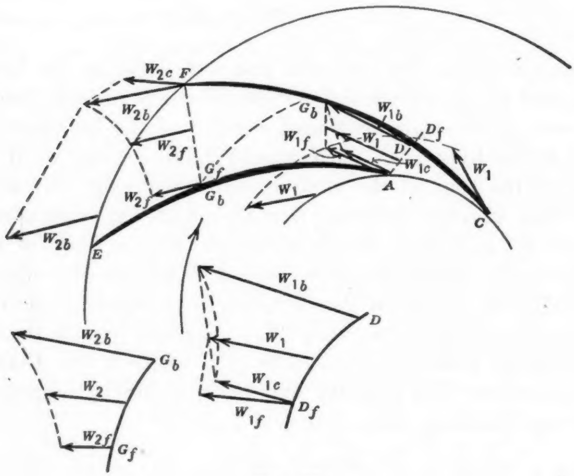

FIG. 22.

The turning moment for every vane is:

$$M = K \frac{N}{G\,z\,n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

in which,

K = a constant;

$\frac{N}{G}$ = the power, in horse power, required by each impeller;

G = the number of the impellers; and

n = the revolutions per minute.

therefore,

$$m = f\,M = K\,f \frac{N}{G\,z\,n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

in which, f denotes the relation between the turning moment, M_1, which corresponds to the part, G-D, of the vane, and the total turning moment, M, for every vane.

It follows that:

$$p_m = K \frac{f}{\varepsilon} \frac{N}{G\,z\,n\,b_m\,(D_2^{\,2} - D_1^{\,2})} = K\,\sigma \frac{N}{G\,z\,n\,b_m\,(D_2^{\,2} - D_1^{\,2})} \ldots (32)$$

The coefficient, σ, may vary between 0.25 and 0.6. Since:

$$N = \frac{\zeta\, Q\, H_m}{550\, \eta_i\, \eta_v\, \eta_m} = \frac{\zeta\, Q\, H_t}{550\, \eta_v\, \eta_m} \qquad (33)$$

$$\psi = K \frac{\sigma}{\eta_v\, \eta_m\, z\, n\, b_m\, (D_2^2 - D_1^2)} \qquad (34a)$$

then,

$$d\, p = \psi\, Q\, H_t \qquad (34b)$$

Solving Equation (34) simultaneously with Equations (23*a*) and (23*b*), respectively:

$$W_2^2 - W_{2b}^2 = 2\, g\, r_2\, \psi\, Q\, H_t \qquad (35a)$$

$$W_{1b}^2 - W_{1c}^2 = 2\, g\, r_1\, \psi\, Q\, H_t \qquad (35b)$$

However, from Fig. 22,

$$W_2^2 - W_{2b}^2 = (W_2 + W_{2b})\,(W_2 - W_{2b}) = 2\, W_{2c}\, 2\,(W_2 - W_{2c}) \qquad (36a)$$

$$W_{1b}^2 - W_{1c}^2 - (W_{1b} + W_{1c})\,(W_{1b} - W_{1c}) = 2\, W_1\, 2\,(W_1 - W_{1c}) \qquad (36b)$$

so that:

$$W_{2c}\,(W_2 - W_{2c}) = \frac{g\, r_2\, \psi}{2}\, Q\, H_t \qquad (37a)$$

$$W_1\,(W_1 - W_{1c}) = \frac{g\, r_1\, \psi}{2}\, Q\, H_t \qquad (37b)$$

From Fig. 21:

$$W_{2c} = \frac{C_{2m}}{\sin \beta_{2c}} = \frac{Q}{\pi\, D_2\, b_2 \sin \beta_{2c}} \qquad (38a)$$

$$W_{1c} = \frac{C_{1m}}{\sin \beta_{1c}} = \frac{Q}{\pi\, D_1\, b_1 \sin \beta_{1c}} \qquad (38b)$$

Combining Equation (38*a*) with Equation (37*a*) and Equation (38*b*) with Equation (37*b*):

$$W_2 = \frac{g\, r_2\, \psi\, \pi\, D_2\, b_2 \sin \beta_{2c}}{2}\, H_t + \frac{Q}{\pi\, D_2\, b_2 \sin \beta_{2c}} \qquad (39a)$$

$$W_1 = \frac{Q}{\pi\, D_1\, b_1 \sin \beta_{1c}} - \frac{G\, r_1\, \psi\, \pi\, D_1\, b_1 \sin \beta_{1c}}{2}\, H_t \qquad (39b)$$

Let:

$$K_u = \frac{g\, r_2\, \psi\, \pi\, D_2\, b_2 \sin \beta_{2c}}{2} \qquad (40a)$$

$$K_1 = \frac{g\, r_1\, \psi\, \pi\, D_1\, b_1 \sin \beta_{1c}}{2} \qquad (40b)$$

then Equations (39) become:

$$W_2 = K_u\, H_t + \frac{Q}{\pi\, D_2\, b_2 \sin \beta_{2c}} \qquad (41a)$$

$$W_1 = \frac{Q}{\pi\, D_1\, b_1 \sin \beta_{1c}} - K_1\, H_t \qquad (41b)$$

Substituting in Equation (18), the values W_2 and W_1 as given by Equation (41):

$$H_t = \frac{u_2^2 - u_1^2}{g} - \frac{u_2}{g}\sqrt{\left(K_u H_t + \frac{Q}{\pi D_2 b_2 \sin \beta_{2c}}\right)^2 - \frac{Q^2}{\pi^2 D_2^2 b_2^2}}$$

$$+ \frac{u_1}{g}\sqrt{\left(\frac{Q}{\pi D_1 b_1 \sin \beta_{1c}} - K_1 H_t\right)^2 - \frac{Q^2}{\pi^2 D_1^2 b_1^2}} \quad \text{..........(42)}$$

Equation (42) is a fourth-power equation of H_t and Q.

When $Q = 0$:

$$H_{to} = \frac{\dfrac{u_2^2 - u_1^2}{g}}{1 + K_u \dfrac{u_2}{g} + K_1 \dfrac{u_1}{g}} \quad \text{..................(43)}$$

but when the discharge is zero, the fundamental equation is, $\frac{u_2^2}{2g}$. Apparently, therefore, there are two values of the head when $Q = 0$; but, in reality, the value, $\frac{u_2^2}{2g}$, corresponds to the condition when $Q + Q_v = 0$, in which, Q_v denotes the portion of delivery, lost in the clearances and the value, H_{0t}, given by Equation (43) applies to the useful delivery, Q. Consequently, there must be a certain skip in the theoretical head of the impeller as shown in Figs. 19 and 23.

With $H_t = 0$,

$$K_a = \frac{n}{60\, g\, b_2}\sqrt{\left(\frac{1}{\sin^2 \beta_{2c}} - 1\right)} - \frac{n}{60\, g\, b}\sqrt{\left(\frac{1}{\sin^2 \beta_{1c}} - 1\right)} \quad \text{..... (44)}$$

and Equation (42) becomes:

$$K_a\, Q - \frac{(u_2^2 - u_1^2)}{g} = 0 \quad \text{.................... (45)}$$

and,

$$Q_0 = \frac{u_2^2 - u_1^2}{K_a\, g} \quad \text{.........................(46)}$$

The graphical representation of Equation (42) is a fourth-order curve in H_t and Q. As Fig. 23 shows, this curve cuts the H_t-axis at a point, P_0, which corresponds in value to H_{0t} and it cuts the Q-axis at a point, P_1, which corresponds to the delivery value, Q_0, given by Equation (46).

This theory explains the value, $\frac{u_2^2}{2g}$, which is so often measured in actual characteristics near the point of zero discharge, and which, until now, was an unexplained feature in the old theory.

In Fig. 23 the part of the curve, P_0-P_2, that is most important is approximately a straight line. Making the assumption that it is a straight line is justified for the sake of greater simplicity and gives results that are in

accord with all the latest hydrodynamic theories. The result is the straight line equation:

$$H_t = \frac{W\,\Gamma\,z}{2\,\pi\,g} = \frac{n\,\Gamma\,z}{g} \quad \text{.....................(47)}$$

in which, Γ denotes vane rotation.

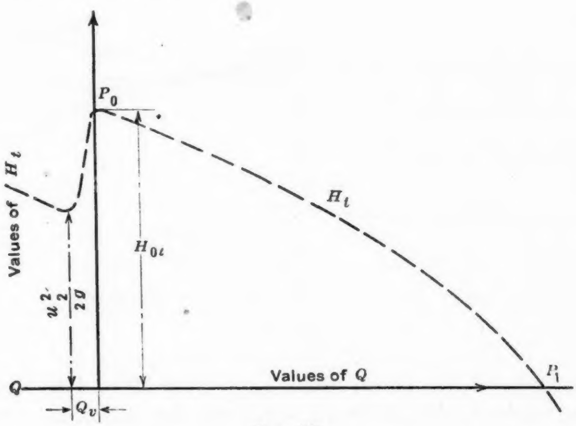

FIG. 23.

It is evident, therefore, that the number of vanes to be provided in turbopumps, blowers, and impellers is limited by practical conditions at two opposite extremes. Considering friction losses in the impeller ducts as the most important factor in the design, it would be desirable to use the least possible number of impeller vanes; but if the cavitation losses were considered most important, it would be necessary to avoid choosing z below a certain value, which may be easily determined by studying the performance of each specific impeller. For radial impellers, these conditions may be expressed, in metric units, as follows:*

$$z\ (\text{minimum}) = 10\,\frac{Q\,H_m}{n\,\eta_t\,b_m\,(D_2^2 - D_1^2)} \quad \text{...............(48)}$$

$$z\ (\text{maximum}) = 30\,D_2 \quad \text{.............................(49)}$$

For an impeller with given values of D_2, D_1, b_2, b_1, β_{2c}, and β_{1c}, the theoretical characteristics, H_t and Q, can be found by assuming different values of z. Plotting these values results in a set of curves similar to Fig. 24(*a*) which refers to a centrifugal pump impeller. By inspection it is apparent in this case that there is little advantage gained by increasing z to more than eight or decreasing it to less than six. Increasing the number of vanes does not cause a proportionate increase in theoretical head and, conversely, decreasing z causes an ever-increasing and disproportionate reduction in head.

The proper choice of the inlet angle, β_{1c}, is also confined, normally, to a narrow range (12 to 25°), because its value is dependent, each time, on the

* "Pompe Centrifughe," by M. Medici, Capitoli III e IX, Hoepli, Milano, Italy, 1927.

best assumption of the value, C_{1m}, the radial component of the absolute speed of the water passing through the inlet. For example, Fig. 24(b) shows the way in which (H_t-Q) values of a centrifugal pump impeller vary for different values of β_{1c} when z and β_{2c} are kept constant at seven vanes and 24° inclination in the outlet vanes.

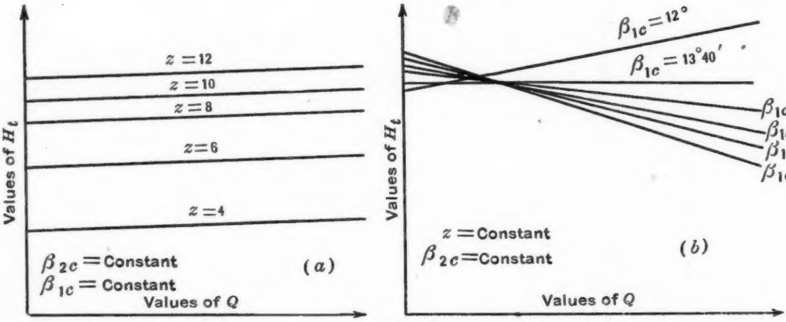

FIG. 24.

On the other hand, β_{2c} could safely vary from 10 to 170° unless practical considerations made it seem advisable to restrict the range, say, between 20 and 150 degrees. Vanes that are inclined too far backward (β_{2c} less than 20°) introduce undue friction losses and those that are inclined too far forward (β_{2c} greater than 150°) promote fluid disturbances and regurgitations within the impeller.

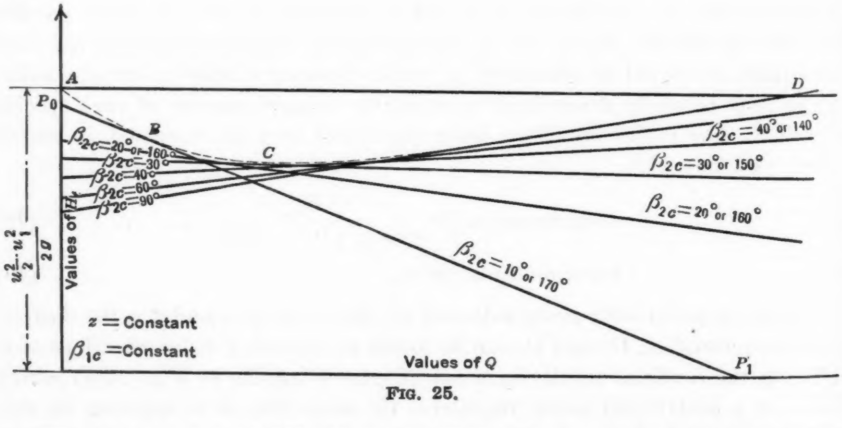

FIG. 25.

Fig. 25 is a set of (H_t-Q)-curves plotted by assuming different values for β_{2c} Values of D_2, D_1, b_1, z, and β_{2c}, are fixed and remain constant in each case. It is apparent that, with Q equal to zero (or nearly zero), the head, H_{0t}, decreases as β_{2c} is increased from 10 to 90°; but as β_{2c} continues to increase from 90 to 170°, H_{0t} increases again at the same rate. In other words, H_{0t} has the same value for β_{2c} = 30° and 150°, for β_{2c} = 10° and 170°, etc. Furthermore, with β_{2c} fixed somewhere between 24 and 32° (depending

on the different types of impeller), the $(H_t\text{-}Q)$-characteristic is parallel to the Q-axis, or the head is constant, theoretically, for varying values of Q. For values outside this range, the head varies as indicated in Fig. 25. According to present theory it seems that the most effective values of β_{2c}, for radial pump impellers, lies somewhere between 24° and 30° and this is in perfect harmony with practical research findings.

The direct development of this theory leads to the establishment of a "non-effective" zone for radial impellers with a finite number of vanes. This zone is shown in Fig. 25 as the area above the dotted line, $A\ B\ C\ D$. Vanes inclined backward (β_{2c} less than 90°), which are considered the best for the development of head according to present-day standards of manufacture, are also shown to be the best according to present theory. With the modern, more complete analysis, theory and practice are perfectly harmonized.

The writer has investigated the performance of a multi-stage pump in order to obtain a definite verification of the present theory and, if possible, to control the values assigned to σ (see Equation (32)). These tests were made in connection with studies of the cavitation phenomenon and suction performance in centrifugal pumps.* Different depressions at the pump suction nozzle were regulated by means of a throttle flap located in the suction pipe. The head and delivery of the pump were measured after each change. The tests were repeated for three values of n: 1 450, 1 600, and 1 750.

Other values were: $D_1 = 0.36$ ft.; $D_2 = 0.82$ ft.; $b_2 = 0.023$ ft.; $b_1 = 0.0492$ ft.; $\beta_{2c} = 30°$; $\beta_{1c} = 17°$; $z = 6$; $G = 3$; P_a (atmospheric pressure) $= 29.8$ in.; water temperature $= 64.5°$ Fahr.; specific weight of water $=$ 62.25 lb. per cu. ft.; and $P_t = 0.83$ in. (water column). The suction pipe was straight and was 5 ft. in length. Packings were very carefully lined in order to introduce a minimum amount of air. The artificially produced suction heads were 23.8, 26.2, and 27.8 ft. The results of the research are shown in Fig. 26. The largest suction heads always cause the most pronounced cavitation, with a corresponding decrease in the quantity of water delivered. Values of σ were computed by use of the formula:†

$$\sigma = \frac{\zeta\, G\, z\, n\, \eta_t\, (D_2^{\,2} - D_1^{\,2})\, b_m \left[\frac{\zeta\, w}{\zeta} \left\{ (P_a - P_t) - \frac{C_{1a}^{\,2} - C_1^{\,2}}{2\, g} \right\} - H_a \right]}{K\, Q\, H_m\, \zeta_w} \quad ..(50)$$

ζ_w is the specific weight of water, found to be 15.56° Fahr. The results of these tests are given in Table 2.

Under the conditions of Test No. 8, which is the normal performance of the pump, $\sigma = 0.25$. Then, from Equation (40*a*),

$$\mathbf{K_u = 0.0856}$$

From Equation (40*b*),

$$K_t = 0.0474$$

$$1 + k_u + k_t = 1.2265$$

* "Untersuchungen über des Verhalten der Saugfähigkeit der Kreiselpumpen," by M. Medici, Heft 1, *Fördertechnik und Frachtverkehr*, 1927.

† Il fenomeno di cavitazione ed i limiti di aspirabilità delle pompe centrifughe," by M. Medici, No. 26, *Il Monitore Tecinco*, 1925.

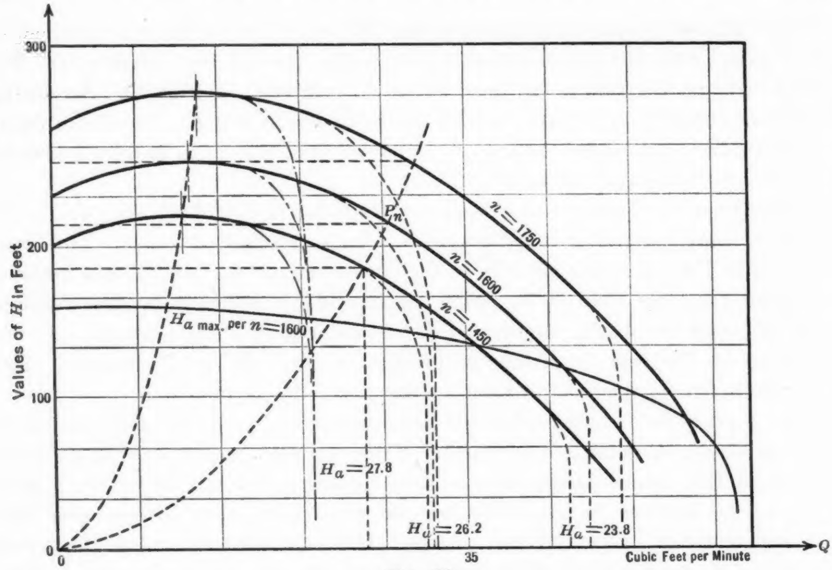

FIG. 26.

TABLE 2.—RESULTS OF TESTS ON A CENTRIFUGAL PUMP.

Test No.	H_a.	n.	Q_e, in cubic feet per minute.	σ.
1	27.8	1 450	21.8	0.364
2	27.8	1 600	21.8	0.346
3	27.8	1 750	21.8	0.318
4	26.2	1 450	31.4	0.305
5	26.2	1 600	32.0	0.3
6	26.2	1 750	32.6	0.25
7	23.8	1 450	44.5	0.27
8	23.8	1 600	46.0	0.25
9	23.8	1 750	49.0	0.24

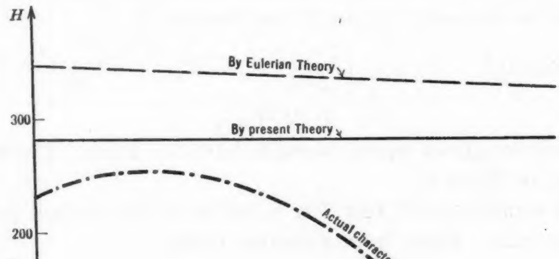

FIG. 27.

$$G\,\frac{u_2^2 - u_1^2}{2\,g} = 348 \text{ ft.}$$

$$G\,H_{0t}'' = \frac{348}{1.2265} = 284 \text{ ft.}$$

From Equation (44),

$$K_u = 5.75 \text{ ft.}$$

From Equation (46),

$$Q_0 = \frac{348}{5.75} = 60.5 \text{ cu. ft. per sec., or } 3\,630 \text{ cu. ft per min.}$$

Fig. 27 shows a comparison of theoretical characteristics (H_t-Q), obtained by the use of the new theory and the Eulerian theory and also shows the actual characteristic curve of a pump investigated when $n = 1\,600$ rev. per min.

A. H. BLAISDELL,* ESQ. (by letter).†—After a careful study of this paper, the writer cannot bring himself to the point of agreeing with the author on the basic assumption which lies at the foundation of the latter's argument. Professor Sherzer states‡ that,

"* * * water being set in rotation in a cylindrical vessel by the action of a paddle or impeller, there is pressure created by centrifugal force which is equal to $\frac{u_a^2}{2\,g}$ and which is part and parcel of the velocity itself. The total head may be regarded as either, but not both".

The fallacy of this idea should not be difficult to comprehend.

In Fig. 28 is shown in cross-section a cylindrical casing, *a*, containing a paddle, *b*, which is attached to a vertical shaft that projects through the upper side of the casing. Assume the shaft and its paddle to be rotating with a uniform angular velocity. If the casing contains a liquid the latter will be forced into rotation and, as Professor Sherzer states, a static pressure is developed due to the centrifugal action; but, contrary to the author's viewpoint, this pressure or potential energy head can be considered, together with the velocity head, as the total energy head imparted to the liquid, if hydraulic losses are neglected.

At the outer edge of the casing are located two piezometer tubes, *c* and *d*. Tube *c* is a static tube with its lower end opening directly into the casing, and Tube *d* has its lower end projecting down into the whirling liquid, the end being turned through 90° with the opening facing directly into the flow. That is, *d* is a Pitot tube. Evidently, with the upper ends of the tubes open to the atmosphere, liquid will rise upward in both, but not to the same heights, as the author's theory indicates. In Tube *c* the liquid will rise to a height, h_a, just sufficient to balance the pressure due to the rotation of the liquid. At Tube *d*, it will be found that the liquid has mounted to a height of H ft., which is equal to twice the height of the liquid in Tube *c*. This is not strange since this tube measures the sum of the static and velocity heads. In order to

* Asst. Prof., Mech. Eng., Carnegie Inst. of Technology, Pittsburgh, Pa.

† Received by the Secretary, January 30, 1928.

‡ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1777.

convince oneself that h_a and H can be properly considered as energy heads, it is only necessary to cut off the tops of the tubes at heights a little less than h_a and H. Liquid will start flowing out from the tops of both tubes and the discharge will continue as long as liquid is supplied to the casing. In other words, the casing and its rotating paddle is a rude sort of pump which is doing the work of lifting liquid through the heights of h_a and H.

FIG. 28.

FIG. 29.

The fact that the pressure developed by the centrifugal action can also be considered as an energy head, may be demonstrated analytically as follows. Let a of Fig. 29 represent the casing of the pump in which liquid is rotating with the uniform angular velocity, ω. Consider an elementary ring of the liquid at a distance, r, from the center of rotation. In order that the shape of the ring may be maintained it is necessary that the centrifugal force developed by the rotation of the ring be counterbalanced by the pressure of the liquid against the outer surface of the ring.

The centrifugal force generated by the rotation of the section, $r\ d\theta\ dr$ (and of unit thickness), will be,

$$dC = r\ \omega^2\ dm = \frac{\gamma}{g}\ r\ d\theta\ dr\ r\ \omega^2$$

in which, γ is the density of the liquid, in pounds per cubic foot. The total pressure acting on the outer wall of this differential section can be expressed as $(r + dr)\ dp\ d\theta$, which is, $r\ d\theta\ dp$, if the product, $dp\ dr\ d\theta$, is neglected. Equating the expressions for centrifugal force and restraining pressure gives,

$$p = \frac{\gamma}{g\ \omega^2} \int_0^R r\ dr = \frac{\gamma}{g}\ \frac{\omega^2\ R^2}{2} = \gamma\ \frac{U_a^{\ 2}}{2\,g}$$

in which, U_a is the linear velocity at the radius, R. Dividing the pressure by the density now provides,

$$\frac{p}{\gamma} = h_a = \frac{U_a^{\ 2}}{2\,g}$$

as the pressure head (or energy) which must be added to the velocity head to obtain the total energy head at a distance, R, from the center of rotation.

With respect to the application of Bernouilli's theorem to the flow of liquids, the writer again finds it necessary to disagree with the author. Strictly speaking, this theorem, which is only a restricted form of the law of conservation of energy, applies only to flows where the motion is steady or stream line and the form of channel or nature of the motivating mechanism which maintains the flow does not come into consideration. In the case of actual liquids, the steady state can only be approximated since the effects of viscosity and boundary friction cause an eddying and turbulent motion where the kinetic energy of translational motion is only a part of that present. For these reasons it is quite necessary to apply the theorem with considerable care to engineering problems where one is dealing with real fluids.

To be sure, the theory of centrifugal pumps as discussed by Professor R. L. Daugherty* has many limitations but these have been recognized and the only excuse for the continued use of this theory has been its simplicity and familiarity as compared to the more intricate analyses obtained by use of higher mathematics. Relatively recent investigations in aerodynamics have led to a new interpretation of the classical hydrodynamics,† making it possible to analyze the fluid action in turbo-machinery with very good approximation to the actual flow phenomena. By revealing what takes place in a blower or pump, these same researches have made it possible to apply the classical theory of centrifugal pumps with satisfactory results.

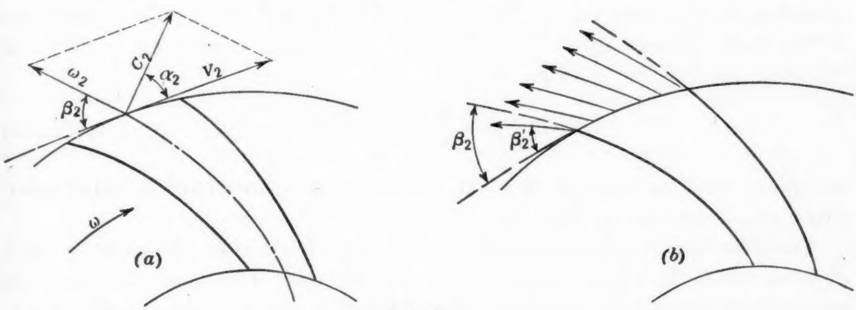

FIG. 30.

The writer believes that Equation (B)‡ is quite satisfactory as it stands. Without doubt the motion of the impeller extends back into the liquid in the suction pipe giving a swirl or corkscrew motion to the approaching mass. The practical effect, in so far as Equation (B) is concerned, is to increase the magnitude of the term, $C_1 \, V_1 \cos \alpha_1$, since under this condition angular momentum is supplied to the liquid before it reaches the impeller. Aside from this direct influence of the impeller, there is (contrary to Professor Sherzer's viewpoint) energy in the liquid which enters the rotor due in most cases (if supply head is negative) to the action of atmospheric pressure. Of course, an equal amount of energy must be furnished to the liquid by the impeller.

* In "Centrifugal Pumps."

† "Hydrodynamics," by Horace Lamb, 4th Edition, 1916.

‡ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1779.

Very careful analytical and experimental work has conclusively shown that the liquid discharges from an impeller with a relative angle, β_2' (Fig. 30 (*b*)), which is less than the one indicated by the customary vector layout (Fig. 30 (*a*)). Moreover, the relative velocity of discharge at a given section of the impeller channel is not constant. For example, at the exit section, the liquid at the rear of the blades discharges with a greater relative velocity than that on the front side. Consequently, the pressure of the flowing liquid is higher in front and less on the rear, etc. However, these facts, when understood, make it possible to continue to use the classical theory.*

MORROUGH P. O'BRIEN,† JUN. AM. SOC. C. E. (by letter).‡—Professor Sherzer's comparison§ of the centrifugal pump to an athlete throwing a hammer is very apt, but not in the manner suggested. The tangential velocity determines the kinetic energy which will propel the hammer and is for this reason the only energy content of interest to the athlete; but there is also present in the rotating system an additional energy due to the stretching of the wire. In a centrifugal pump, the tension in the wire is replaced by the centripetal force exerted by the pump casing.

Probably the simplest manner of considering the shut-off condition in a centrifugal pump is to regard it as a forced vortex surrounded by a liquid moving irrotationally.‖ The condition of irrotational motion in the surrounding field is that the product of the velocity and the radius of curvature of the path at any point is a constant. For such a rotating system, the equation of the free surface is:

$$Z = -\frac{W^2\,a^4}{2\,g\,r^2} + \frac{W\,a^2}{g} \qquad (51)$$

in which, W is the angular velocity of the forced vortex and the other quantities are as shown in Fig. 31.

Consider the liquid contained between two horizontal planes at a small distance apart, in which a forced vortex is present. Equation (51) gives the pressure difference between the center of rotation and a point at a distance, r. It is seen that for $r = \infty$, the pressure is $\frac{W^2\,a^2}{g}$ or $\frac{V^2}{g}$, and that at $r = a$, it is $\frac{W^2\,a^2}{2\,g}$ or $\frac{V^2}{2\,g}$.

For any value of r, the vertical distance from the z-line to the value of z at $r = \infty$, represents the velocity head still present in the liquid. As the casing of a pump is somewhat larger than the impeller, but is never of infinite

* See "A Method of Analyzing the Performance Curves of Centrifugal Pumps," by J. Lichtenstein, *Transactions*, Am. Soc. Mech. Engrs., Vol. 49, November, 1927, p. 1269 (Synopsis); also, "The Hydrodynamic Theory of Turbines and Centrifugal Pumps," by Dr. Ing. Bruno Eck, *Engineering* (Lond.), January 22, 1926; and, also "Kolben- und Turbo-Kompressoren," by P. Osterlag, Julius Springer, Berlin, Germany.

† Toledo, Ohio.

‡ Received by the Secretary, February 23, 1928.

§ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1778.

‖ "The Mechanical Properties of Fluids," D. Van Nostrand, 1924.

dimensions, the pressure head at shut-off may be greater than $\frac{V^2}{2\,g}$, but can never be $\frac{V^2}{g}$.

When liquid is being discharged through the forced vortex and surrounding irrotational field, the determination of the pressure distribution is greatly complicated by the fact that the angular velocity of any fluid particle is not

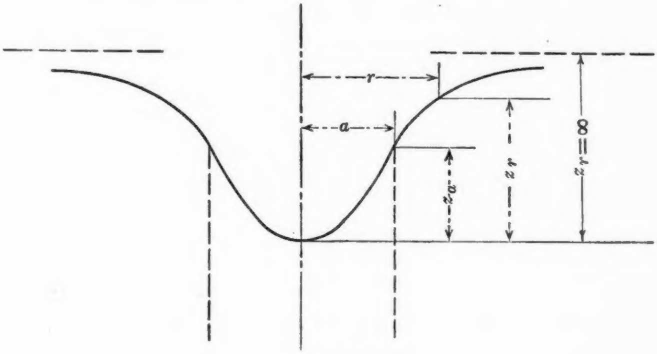

FIG. 31.

the same as that of the impeller, except for radial blades, and by the fact that the radius of curvature to be used in computing the centrifugal force is not the distance from the particle to the axis of rotation of the impeller. The equation may be obtained by combining Corioli's law for the composition of accelerations with Bernoulli's general equation, which includes the change in energy content along a stream tube.* The resultant equation is the Eulerian equation found in most texts.

Professor Sherzer's data showing that the pump may be considered as a series of three orifices,† really indicate that a velocity head corresponding to the velocity that is normal to the first two areas is lost in impact or friction. Experiments by Stanton‡ showed that the efficiency of vortex chambers very often does not exceed 10%, a result that roughly agrees with the indication of losses shown by these experiments. As to the third orifice, at the entrance to the discharge line, the velocity head present at this point must also be present at the point at which the pressure is measured. Since there are no dimensions of the experimental pumps given, it is impossible to consider the data in the light of accepted theories. From the general description, the new design seems to correspond to some of the earliest types of centrifugal pumps. Other references to this subject may be found in theories proposed by Mr. A. H. Gibson.§

* "Om Bernoulli's Ekvationer och, Ansatser Till en Komplettering av Turbinteorien," by Hjalmar O. Dahl, *Teknisk Tidskrift*, February 17, 1923, p. 13.

† *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1789.

‡ *Proceedings*, Inst. of Mech. Engrs., 1903.

§ "Hydraulics and Its Applications," by A. H. Gibson, D. Van Nostrand, 1925.

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PAPERS AND DISCUSSIONS

THE HYDRAULIC DESIGN OF FLUME AND SIPHON TRANSITIONS

Discussion*

By Messrs. F. Theodore Mavis, W. H. R. Nimmo, and Carl Rohwer.

F. Theodore Mavis,† Assoc. M. Am. Soc. C. E. (by letter).‡—The flow of water through the more common types of transition sections can be calculated quite accurately by the methods which the author has discussed. Occasionally, one sees an unusual type of transition section which cannot be investigated by the analytical methods available at present, and the studies must be carried on almost entirely in the hydraulic laboratory. It may be desirable to rely, in part at least, on experimental methods if the transition sections are of unusual shape or if the velocities exceed the critical velocity.

One of the most unique examples of tunnel inlet transitions which has come to the writer's attention is that for the proposed flood diversion tunnel at Nürnberg, Germany. This tunnel is designed to divert 12 400 cu. ft. per sec. from the Pegnitz River in times of extreme floods in order to protect the "Alstadt", which is one of the oldest and most picturesque towns in Germany. Just above the tunnel inlet 2 800 cu. ft. per sec. is to be diverted into power canals and into the old river channel through the city.

Under the direction of Professor Theodore Rehbock, extensive tests for this project were made in the Hydraulic Laboratory of the Technische Hochschule in Karlsruhe on eleven models of inlet transitions, two models of outlet transitions, and three combined models of inlet and outlet connected by a short piece of tunnel. Figs. 24, 25, and 26 show sketches of nine of the proposed inlet structures which were studied in the laboratory. All linear dimensions are in meters and all velocities in meters per second

* Discussion of the paper by Julian Hinds, M. Am. Soc. C. E., continued from December, 1927, *Proceedings*.

† Urbana, Ill.

‡ Received by the Secretary, February 15, 1928.

in the full-sized structures as indicated by tests of the models. Most of the models were made to a scale of 1:100.

The inlet transition which was proposed in the original plans is essentially a sector of a right circular cone with side slopes of 1 on 9 and a central angle of about 70° between retaining walls. Fig. 24 (*a*) shows a plan of this inlet and Fig. 24 (*b*) a longitudinal section with the water sur-

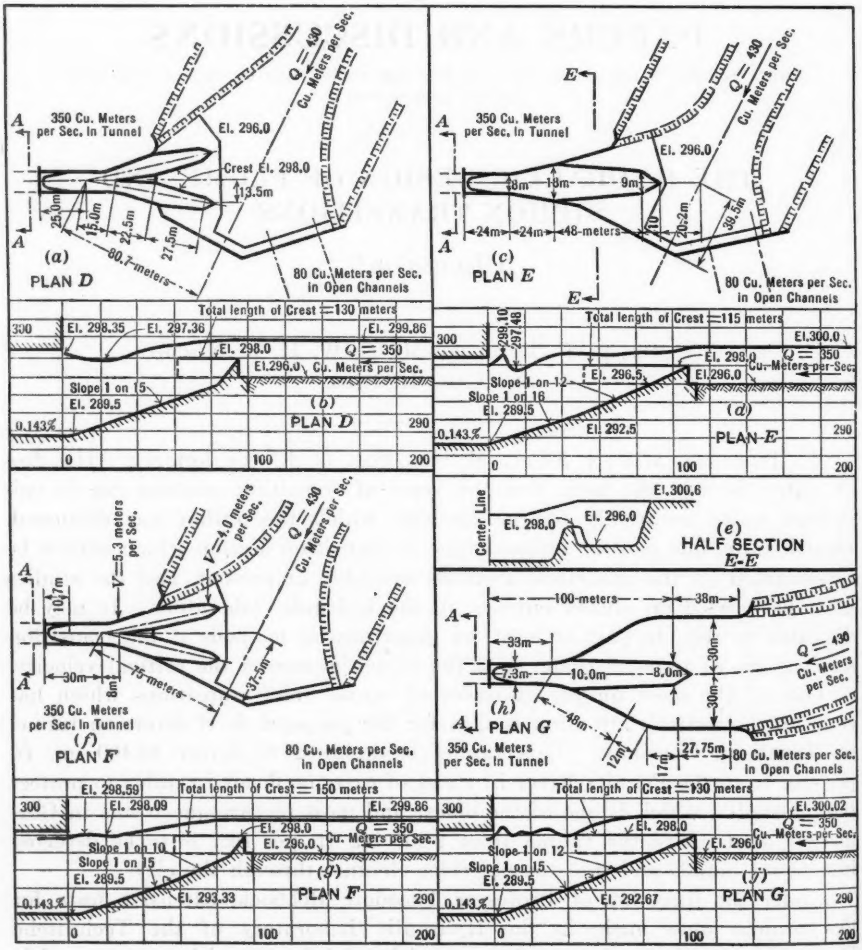

FIG. 24.—PLANS FOR DIVERSION OF PEGNITZ RIVER FLOODS AT NÜRNBERG, GERMANY.

face in the prototype determined from the model experiments. The flow through this inlet section was marked with high waves which lapped the roof of the tunnel. Under these conditions of flow there is grave danger of air trouble which has already been discussed by the author.* The changes in the velocity of flow through the section were not smooth and gradual; there

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1828.

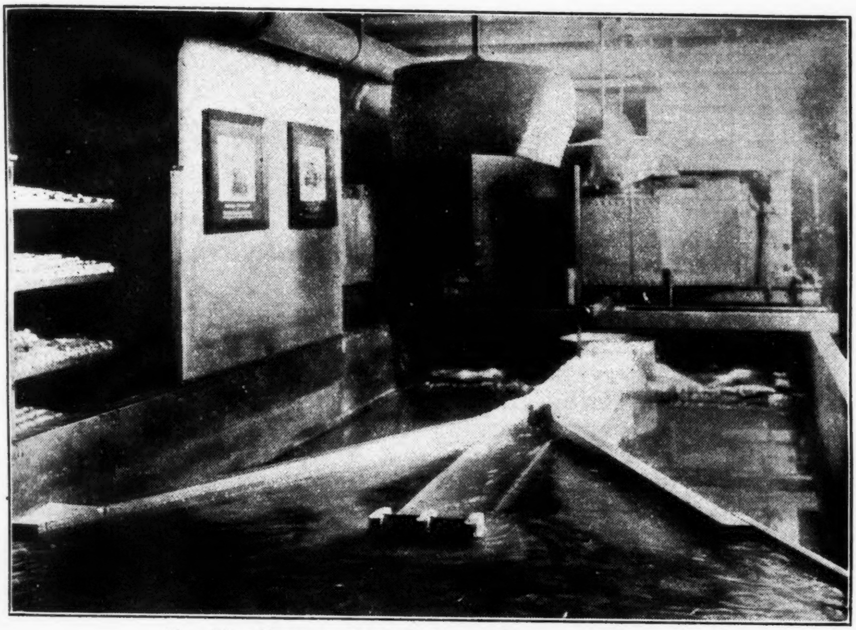

FIG. 25.—VIEW OF FLOW THROUGH MODEL FOR PLAN *A*.

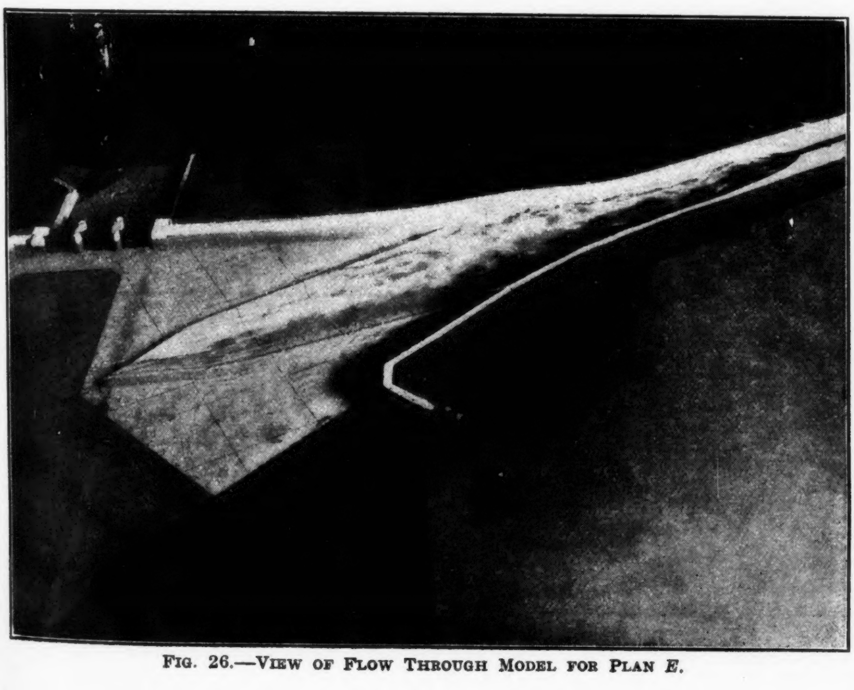

FIG. 26.—VIEW OF FLOW THROUGH MODEL FOR PLAN *E*.

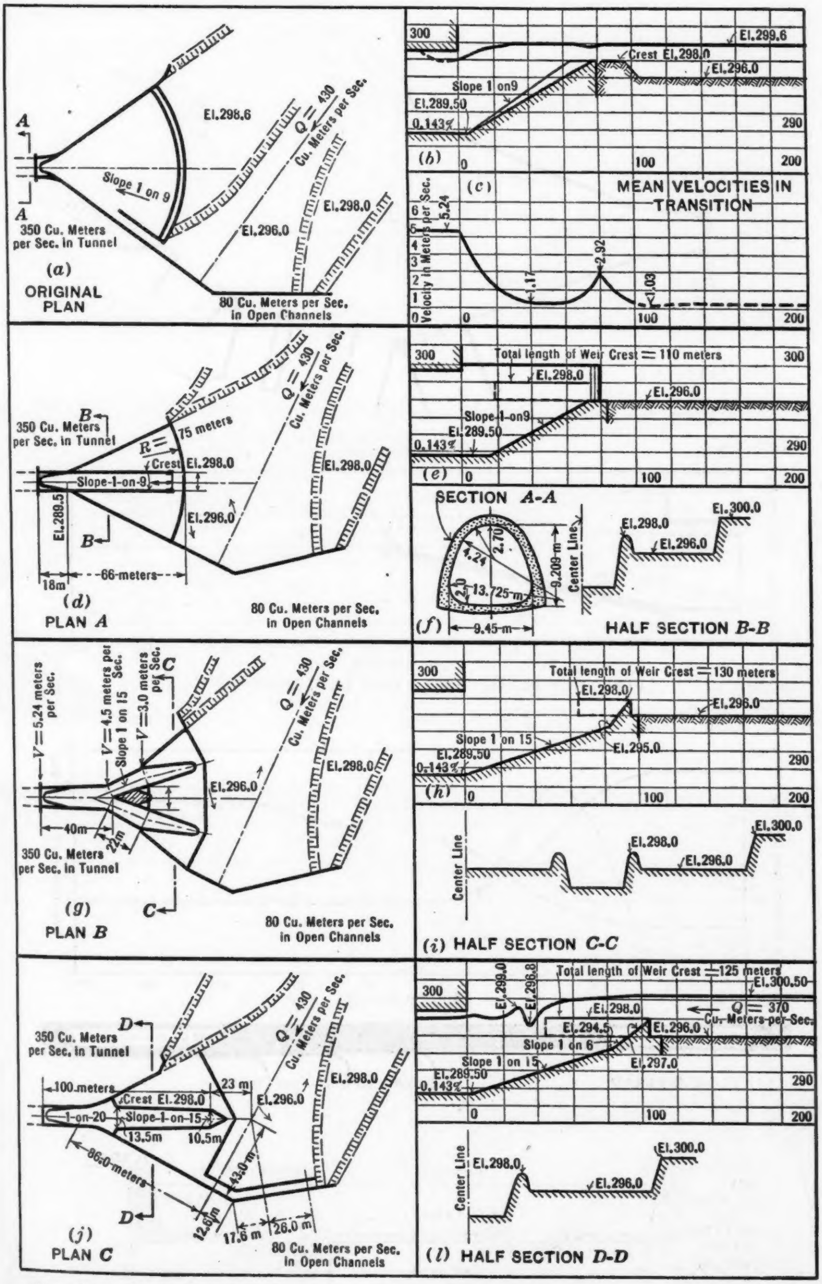

FIG. 27.—PLANS FOR DIVERSION OF PEGNITZ RIVER FLOODS AT NÜRNBERG, GERMANY.

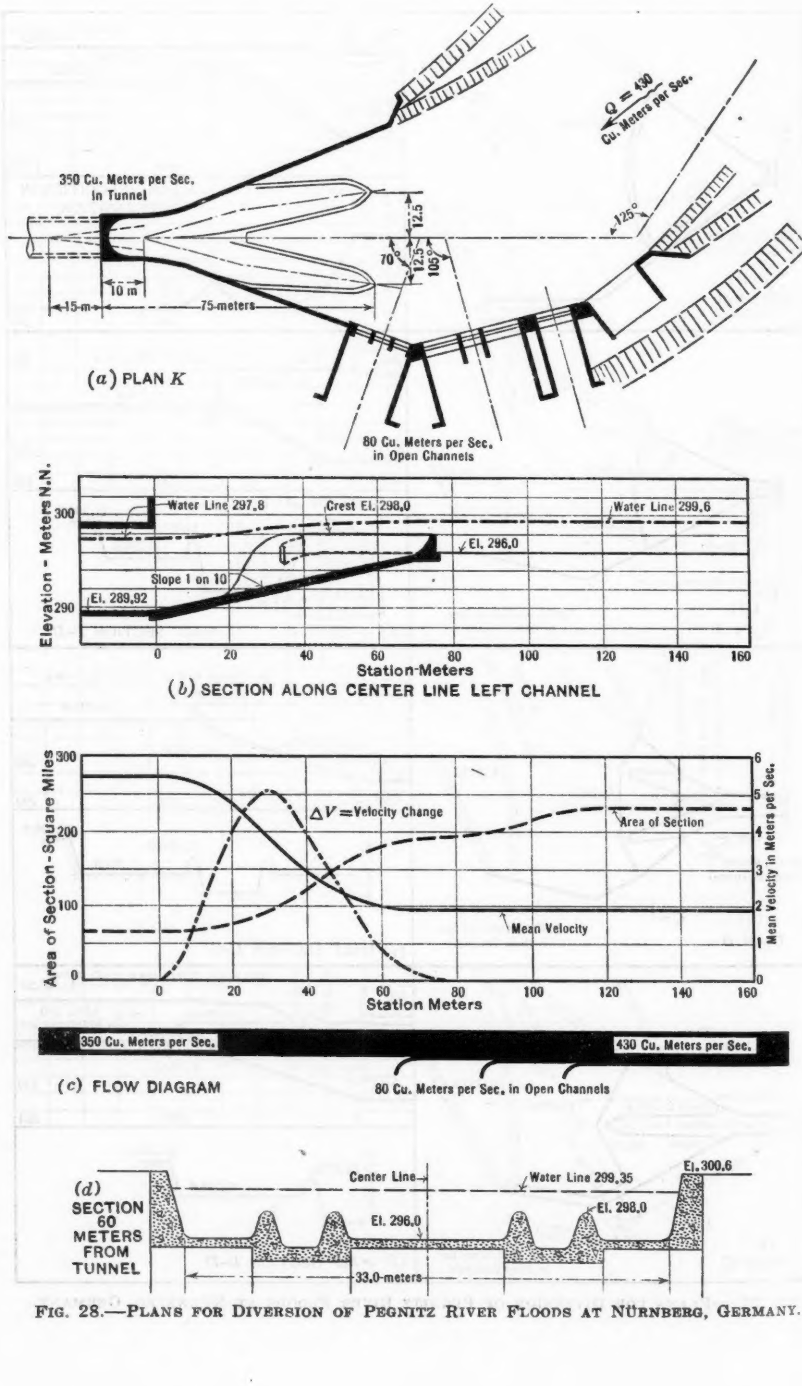

FIG. 28.—PLANS FOR DIVERSION OF PEGNITZ RIVER FLOODS AT NÜRNBERG, GERMANY.

FIG. 29.—VIEW OF FLOW THROUGH MODEL *H*.

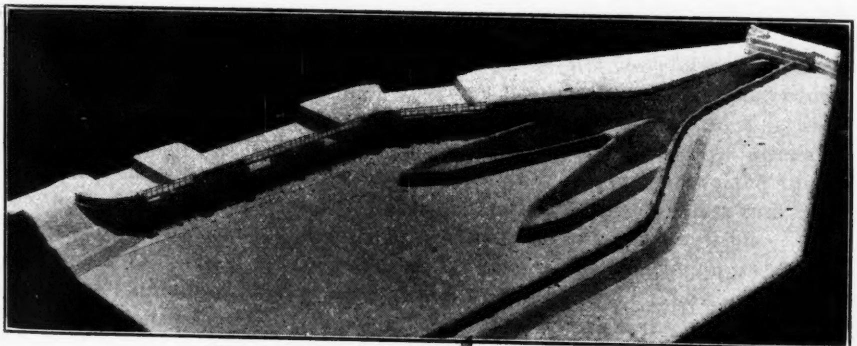

FIG. 30.—VIEW OF MOST SATISFACTORY MODEL.

FIG. 31.—VIEW OF FLOW THROUGH MODEL *J*.

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were abrupt changes near the entrance to the tunnel and below the sill or weir crest where the flow passed the critical stage. The velocity changed from 1.03 m. per sec. in the approach channel to 2.92 m. per sec. at the weir crest. A hydraulic jump occurred below the weir. The velocity changes to 5.24 m. per sec. in the tunnel, with a minimum of 1.17 m. per sec. in transition. The slope of the water surface changes from 1 on 10 in transition to 1 on 700 in the tunnel. These velocity changes are shown in Fig. 24 (*c*).

The subsequent plans provided that the bottom of the river channel was to be extended to the full width of the inlet transition section. Plan *A*, Fig. 24 (*d*), (*e*), and (*f*), provided for a collecting trough 35 ft. wide and 180 ft. long, with sides or sills 6.6 ft. above the bottom of the approach channel. The bottom of this trough was sloped upward (1 on 9) from the bottom of the tunnel to the level of the river bed, but the bottom of the transition section outside the trough walls was level and at the elevation of the river bottom (Elevation 296.0). The flood water spilled over the sides of this trough and was then directed into the tunnel.

The flow through the inlet of Model *A* was not satisfactory because, although there was a smooth flow in the approach channel, strong eddies were formed near the point where the sills of the trough joined the side walls of the transition section. These eddies restricted the effective discharge area near the tunnel mouth and caused a high wave just above the tunnel entrance. Fig. 25 shows the flow through the model during a test run. The eddies and the wave in the approach to the tunnel are clearly visible.

Plan *B* (Fig. 24 (*g*), (*h*), and (*i*)), shows a double trough section in which each trough is tapered toward the up-stream end. The flow through this transition was much smoother than that through the one originally proposed or through the transition of Model *A*. The waves that form at the up-stream end of the "ships" can be eliminated by pointing the "prows".

Plan *C* (Fig. 24 (*j*), (*k*), and (*l*)), Plan *E* (Fig. 27 (*c*), (*d*), and (*e*)), and Plan *G* (Fig. 27 (*h*) and (*i*)), are single-trough transitions which have the same general shape but different sized troughs. The flow in the model for Plan *E* was much smoother than in that for Plan *C*, but was not considered satisfactory. In Plan *G* the flow was smoother than in Plans *C* or *E*, with strong eddies near the junction of the weirs and side walls. Fig. 26 shows the flow through the inlet of Model *E* during a test run. It will be noted that, although the flow is smoother than that shown in Fig. 25, it is not wholly free from eddies and that a wave is formed in the transition section just above the entrance to the tunnel.

Plan *D* (Fig. 27 (*a*) and (*b*)), Plan *F* (Fig. 27 (*f*) and (*g*)), Plan *K* (Fig. 28), and Plan *H* (Fig. 29) provide for double-trough transitions similar to Plan *B*. In the model for Plan *D* there was observed a smooth flow and no large waves. The troughs of Plan *K* are 50 ft. shorter than those of Plan *H* (Fig. 29) and the direction of flow into the transition is more nearly axial in the latter than in the former plan. The flow through Model *H* (Fig. 29) was very smooth and there were no appreciable waves or eddies in the transition section for the maximum rate of discharge corresponding to 12 400 cu. ft. per sec. Fig. 30 shows a model of the inlet, which was

finally recommended as the most satisfactory one of those studied in the laboratory.

Fig. 28 (*c*) shows the cross-sectional areas, mean velocities, and velocity changes in the transition of Plan *K* and Fig. 28 (*b*) shows the water surface determined from the tests. It is interesting to compare these with Fig. 24 (*b*) and (*c*), which show the water surface and mean velocities in the transition section originally proposed. Remembering that sudden changes in velocity are accompanied by energy transformations and shock losses, it is not surprising to find that the flow in the model of the inlet shown in the original plan was not as smooth as that in the model of the inlet shown in Plan *K*.

Fig. 31 shows the flow in the model of Plan *J*, which is a modification of Model *A*. Crystals of potassium permanganate were scattered in the channel to show the direction of flow of the bottom water filaments and produced the streaks which are shown in the diagram. The flow just above the mouth of the tunnel is near the critical velocity and there is an abrupt drop in the water surface just above the portal. The maximum in the transition section of Model *J* represented a velocity of 31 ft. per sec. in the prototype while the maximum velocity in the transition section of Plan *H* (Fig. 29) represented a velocity of 19 ft. per sec.

It is said that although the single-trough sections were more satisfactory than the transition proposed in the original plan, the heads required to produce a maximum discharge of 12 400 cu. ft. per sec. were greater than those required to produce the same discharge in the double-trough sections. The waves were smaller and the eddies were weaker in the double-trough models than in the single-trough models.

The transition which was recommended below the tunnel outlet is 450 ft. long, 200 ft. wide at the lower end, and slopes upward from the bottom of the tunnel to the canal, which is at a grade 24.6 ft. above the bottom of the tunnel.* The tests conducted for a discharge corresponding to 12 400 cu. ft. per sec. indicated that there was practically a complete recovery of head although the velocity was reduced from 17.4 ft. per sec. to 6.9 ft. per sec. The observed water surface in the model showed a very satisfactory agreement with that calculated by methods similar to those discussed by the author.

The writer is indebted to Professor Rehbock, of the Technische Hochschule, in Karlsruhe, for permission to present this abstract of reports† and test data in his personal files and for the photographs presented herewith.

W. H. R. Nimmo,‡ Assoc. M. Am. Soc. C. E. (by letter).§—The valuable paper presented by the author is of special interest to the writer who, during 1919, was in charge of the construction of a diversion project of 450 sec-ft. capacity, involving a decelerating transition from a concrete flume to a canal excavated in earth. The properties of the flume were: Grade of floor,

* See "Die Wasserbaulaboratorien Europas", V. D. I. Verlag, Berlin, 1926, p. 108.

† Th. Rehbock, "Gutachten über den Entwurf für die Regulierung der Pegnitz zur Beseitigung der Ueberschwemmungen in Nurnberg," May, 1914; "Nachtrag-Gutachten, etc.," December 20, 1914, with supplementary reports of June 21, 1915, and February 16, 1916.

‡ Chf. Draftsman, Main Roads Comm., Brisbane, Queensland, Australia.

§ Received by the Secretary, February 23, 1928.

0.00303%; width, 9 ft.; sides, vertical; normal depth of water at full capacity, 5 ft.; average velocity, 10 ft. per sec.; velocity head, 1.56 ft.; and Kutter's $n = 0.015$. The corresponding properties for the canal were: Grade, 0.000757%; bottom width, 24 ft.; slope of sides, 1 on 1.25; normal depth, 4.32 ft.; average velocity, 3.5 ft. per sec.; velocity head, 0.19 ft.; and Kutter's $n = 0.026$.

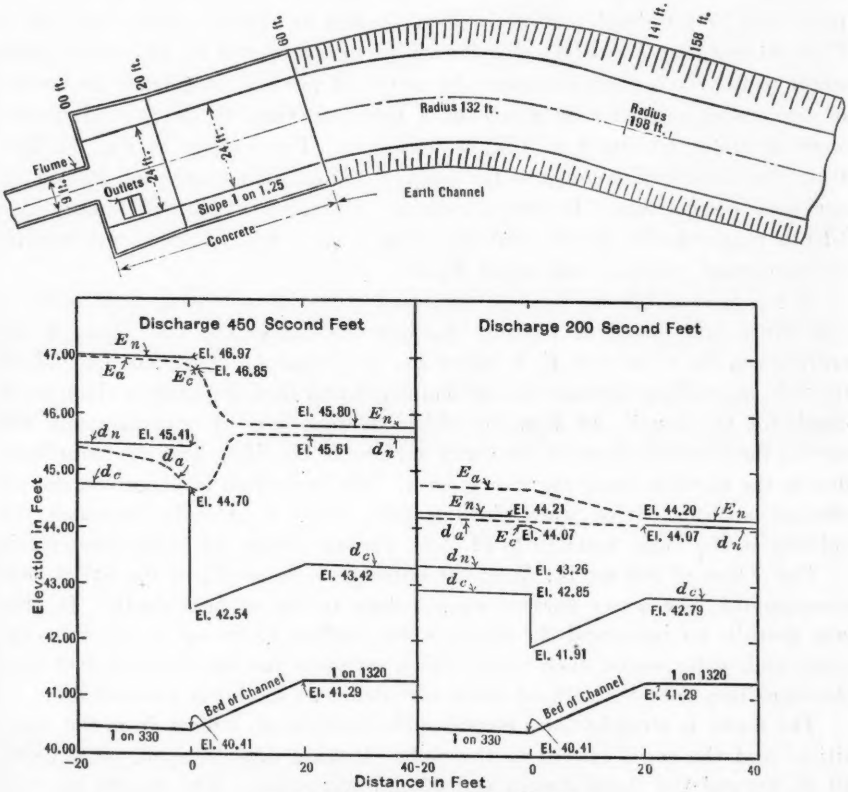

FIG. 32.—PLAN OF TRANSITION.

The excavation was in firm clayey soil which would permit of the building of warped side slopes in concrete without back forms. In order to avoid turbulence, the writer proposed to construct a warped transition, 132 ft. in length, allowing for a loss of head of 0.17 ft., or 13% of the total change in velocity head from flume to canal. This length of transition would have been more than double the minimum length allowed by the author's rule,* but the whole distance need not have been in concrete. The allowance for regain of head was perhaps rather liberal, but it was intended also to have slightly increased the free-board on the flume for a short distance in case the full amount of regain was not realized. It was also proposed that the area of

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1817.

the cross-section occupied by water should vary proportionally to the change in kinetic energy. In view of the evidence presented by the author, it seems that the proposed transition would have operated satisfactorily.

The Department responsible for the design of the transition, however, held the view that it would not be possible to regain any head and, therefore, a transition—or rather a stilling-pool—was built in accordance with a design supplied to the writer. The transition as actually constructed is shown in plan, and longitudinal section in Fig. 32 and is shown empty and full in Figs. 33 and 34. To satisfy riparian rights provision was made for delivering water to the river, when necessary, by means of pipes leading from the bottom of two outlet pits built in the floor of the transition, the horizontal dimensions of each pit being 5 ft. 9 in. by 3 ft. 0 in. The sections in Fig. 32 illustrate the theoretical conditions for flows of 450 cu. ft. per sec. and 200 cu. ft. per sec., respectively. In these sections, d denotes depth; E, energy line (depth plus velocity head); and the suffixes, n, c, and a, refer, respectively, to the normal, critical, and actual depth.

For a flow of 450 sec-ft., it is seen that a considerable drop in the energy line must take place in passing through the transition, but since at the entrance to the transition E_c is below E_n, the actual water surface will follow the line, d_a, passing through the critical depth and then jumping to the normal depth for the canal. In Fig. 34, which shows a flow of approximately 450 sec-ft., the drawing down of the water surface in the flume and the turbulence due to the ensuing jump are clearly seen. The transition is, therefore, not an efficient stilling device when flowing full, since it actually increases the velocity in the flume from 10 to 11.75 ft. per sec. before the jump takes place.

For a flow of 200 sec-ft., E_c at the entrance is below E_n at the outlet and, consequently, the water surface cannot drop to the critical depth. In this case there is no jump and the actual water surface backs up in the flume on some back-water curve, such as d_a, which accounts for the observed fact that the transition operates without great turbulence at the lower rates of flow.

The flume is straight for a considerable distance up stream from the transition, and the canal curves to the right (looking down stream) at a point 40 ft. beyond the down-stream end of the transition. The stream of high velocity persists with small ripples for several hundred feet down stream, but as yet there has been no erosion of the banks of the canal. For some distance below the flume outlet, the water at the sides of the channel flows up stream and forms pronounced eddies in the corners adjacent to the outlet of the flume. One would expect that the effect of the curve in the canal would be to cause the high velocity stream to diverge to the left (looking down stream) but in Fig. 34 it is seen diverging to the right. The writer observed this on the first occasion on which the canal was filled, and thought it might be due to the effect of the outlet pits, which would absorb energy in the eddy on that side, thus causing it to become smaller than the eddy on the other side. After a trial run the flow was shut off, but the next time the headgates were opened the high velocity line diverged to the left. Since then the

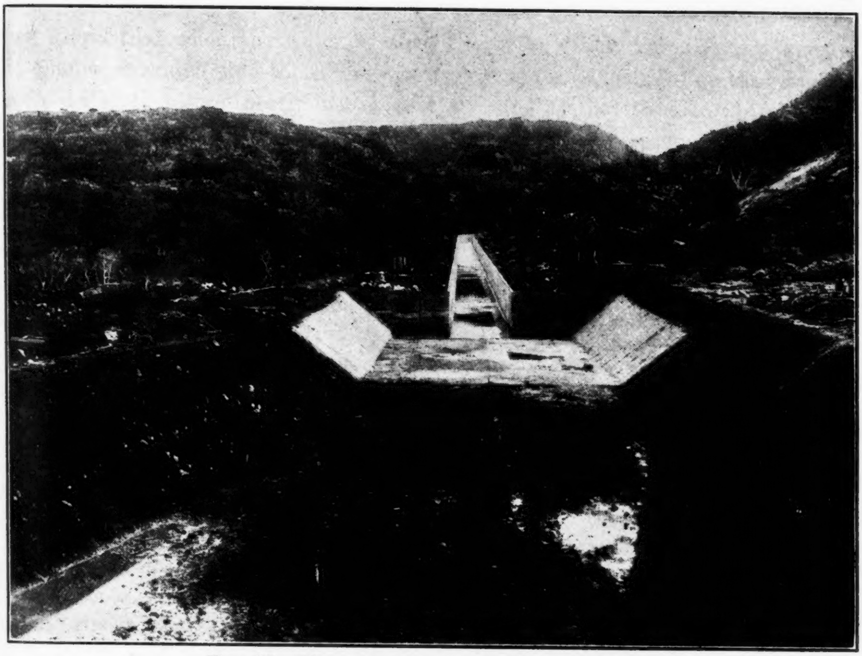

FIG. 33.—VIEW OF TRANSITION WITHOUT FLOW.

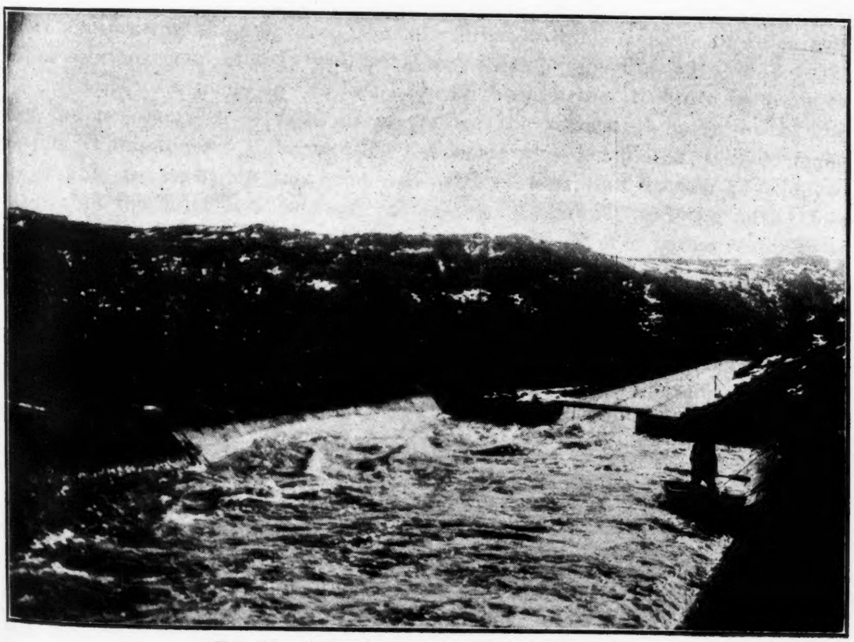

FIG. 34.—VIEW OF TRANSITION IN OPERATION.

writer has observed the action of the transition on numerous occasions and has found that when the water is turned into the empty canal it is a matter of chance to which side the high velocity stream is deflected, but that having been deflected to one particular side, it remains at that side until the water is again shut off. In the cases cited by the author the divergence may be due to curvature, but in this case it is not. E. W. Lane, M. Am. Soc. C. E., has observed similar phenomena in experiments with contractions.*

Carl Rohwer,† Assoc. M. Am. Soc. C. E. (by letter).‡—The design of transitions is an important problem in hydraulics, and the data assembled by the author and the method of design that he has devised are valuable contributions to the Engineering Profession and particularly to the engineers interested in irrigation. However, as the author indicates,§ no satisfactory method of computing the transition losses has as yet been developed, and, consequently, even after a transition is carefully designed, the success of its operation can only be determined by trial.

A study of the experimental data submitted by the author shows that very little loss of head actually occurs in transitions when the velocity is increased, and, apparently, there is no correlation between the suddenness of the transition and the loss in head. The question naturally arises whether the complicated inlets shown by the author are justified when the simpler structures seem equally efficient. Much greater losses arise through structures where the velocity is decreased and the loss increases with the abruptness of the transition; but very little, if any advantage is shown by the bell-shaped outlets, which are certainly more difficult to construct.

In calibrating the improved Venturi flume,‖ which consists of inlet and outlet sections connected by a short throat section (see Fig. 35(*a*) and (*b*)), observations were made on the loss in head through the structure in order to find the net sacrifice of head necessary for the successful operation of the device. The gauges for determining the heads were not placed so as to make it possible to segregate the inlet and outlet losses, but it was possible to determine the inlet loss between the points, M and N, and the outlet loss between the points, P and R. The losses are based on staff-gauge readings at M and R (Fig. 35), and hook-gauge readings at N and P for the flumes with a throat width of from 1 to 8 ft. For the 10-ft. flume, the losses are based on readings with an engineer's level and rod on the water surface. The hook-gauge readings are reliable, but errors are possible in the staff-gauge and level readings which might affect the results materially. Representative results of these observations are shown in Table 5,¶ which is based on tests made to

* "Experiments on the Flow of Water Through Contractions in an Open Channel," *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 1207.

† Associate Irrig. Engr., Div. of Agricultural Eng., U. S. Dept. of Agriculture, Fort Collins, Colo.

‡ Received by the Secretary, February 27, 1928.

§ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1806.

‖ "The Improved Venturi Flume," by R. L. Parshall, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 89, (1926), p. 841.

¶ Prepared from data of Irrigation Investigations of the Div. of Agricultural Eng., Bureau of Public Roads, U. S. Dept. of Agriculture, co-operating with the Colorado Agricultural Experiment Station.

determine the free-flow discharge. In Tables 5 and 6 the author's terminology is used, as follows:

Q = discharge, in cubic feet per second.
V = mean velocity, in feet per second.
Δh_v = change in velocity head.
E = entrance loss, in percentage of Δh_v, measured between Points M and N.
O = outlet loss, in percentage of Δh_v, measured between Points P and R.

E and O include friction losses.

The results of these observations indicate that although there is an abrupt transition at the inlet end of the flume, very little loss occurs in this section and there is no relation between the suddenness of the transition and the loss in head. The losses are, however, somewhat greater than those reported by Mr. Hinds.

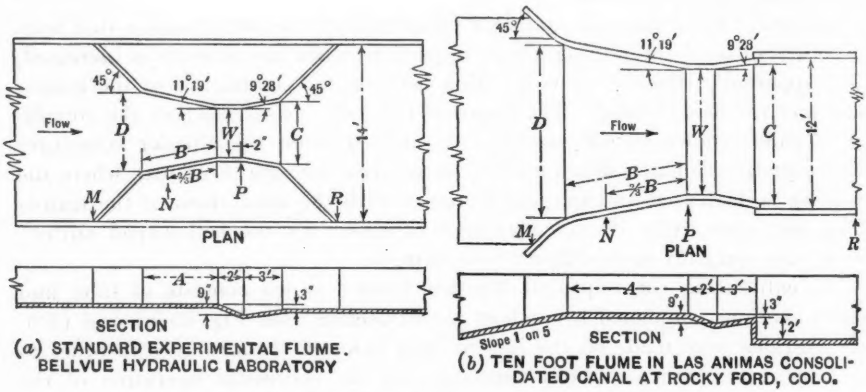

FIG. 35.—SECTION OF IMPROVED VENTURI FLUME.

Outlet losses are, to a great extent, determined by the elevation of the water surface at the lower end of the transition, and, consequently, a wide variation in the percentage recovery is possible, regardless of the correctness of the design of the structure. In calibrating the Venturi flume under free-flow conditions, it was necessary to keep the down-stream depth of the water low enough so that it would not back the water into the throat of the flume and raise the head above the condition of maximum flow. At this point the down-stream depth may vary considerably without materially affecting the condition of maximum flow; consequently, some of the tests chosen (see Table 5) do not show as great a recovery in head as would have been possible if the down-stream depth had been increased. However, the outlet losses of the flumes with throat widths of from 1 ft. to 8 ft. (which were tested under similar conditions in the laboratory) decrease, in general, as the abruptness of the transition decreases. The 10-ft. flume, which is an exception to the rule, is a field installation; as shown in Fig. 35(b), there is a vertical drop of nearly 2 ft. at the down-stream end of the structure which, in addition to the

TABLE 5.—ENTRANCE AND OUTLET LOSSES THROUGH IMPROVED VENTURI FLUMES OF VARIOUS SIZES FOR DIFFERENT FREE-FLOW DISCHARGES.

Test No.	STANDARD DIMENSIONS, IN FEET. (SEE FIG. 5(*a*)).						FLOW CHARACTERISTICS.					
	W.	*A.*	⅔ *A.*	*B.*	*C.*	*D.*	*Q.*	Submergence.*	*V* (canal to flume).	*E.*	*V* (flume to canal).	*O.*
6 481	1	4.50	3.0	4.41	2	2.77	15.13	65.3	0.423 to 2.82	16.5	6.67 to 0.429	135
6 491							10.00	62.5	0.363 to 2.43	22.2	5.40 to 0.384	91
6 488							7.48	76.9	0.325 to 2.16	14.1	3.95 to 0.317	121
6 434	2	5.00	3.33	4.89	3	3.96	33.0	71.1	0.85 to 3.88	13.4	6.65 to 0.87	203
6 438							25.03	65.8	0.78 to 3.55	16.0	6.09 to 0.82	305
6 445							14.93	71.3	0.64 to 2.93	0.0	4.19 to 0.66	67
6 386	4	6.00	4.00	5.88	5	6.35	55.90	69.8	1.57 to 4.40	19.0	6.25 to 1.69	172
6 372							35.66	65.1	1.36 to 3.79	20.5	5.07 to 1.52	27.5
6 399							15.18	78.0	0.96 to 2.62	21.5	2.55 to 1.05	0
6 339	6	7.00	4.67	6.86	7	8.75	49.50	71.7	1.97 to 3.88	34.5	4.56 to 2.18	28.0
6 353							30.06	70.3	1.64 to 3.21	42.0	3.35 to 1.86	0
6 359							20.12	75.3	1.42 to 2.77	22.7	2.43 to 1.44	0
6 336	8	8.00	5.33	7.84	9	11.06	57.96	64.1	2.62 to 3.95	0	4.48 to 3.08	24.4
6 513							35.08	72.7	2.16 to 3.24	11.1	2.99 to 2.32	0
							REFER TO FIG. 5(*b*).					
†	10	9.00	6.00	8.82	11	13.53	126.12	67.5	2.20 to 4.92	6.7	7.65 to 2.88	93
†							90.73	51.3	1.89 to 4.50	0	6.34 to 2.01	187

* Submergence based on the ratios of the upper and lower heads.

† Special tests.

TABLE 6.—ENTRANCE AND OUTLET LOSSES THROUGH IMPROVED VENTURI FLUMES OF VARIOUS SIZES OF DIFFERENT SUBMERGED-FLOW DISCHARGES.

Test No.	STANDARD DIMENSIONS, IN FEET. (SEE FIG. 5(*a*)).						FLOW CHARACTERISTICS.					
	W.	*A*.	⅔ *A*.	*B*.	*C*.	*D*.	*Q*.	Submergence.*	*V* (canal to flume).	*E*	*V* (flume to canal).	*O*.
6 490	1	4.50	3.00	4.40	2	2.77	7.48	92.3	0.246 to 1.60	0	2.82 to 0.230	27.4
6 447	2	5.00	3.33	4.89	3	3.96	14.67	94.2	0.471 to 2.04	0	2.68 to 0.437	27.5
6 384	4	6.00	4.00	5.88	5	6.35	43.42	90.5	1.27 to 3.40	0	3.93 to 1.23	0
6 341	6	7.00	4.67	6.86	7	8.75	50.96	88.2	1.86 to 3.59	20.5	3.78 to 1.84	0
6 298	8	8.00	5.33	7.84	9	11.14	55.57	91.0	2.30 to 3.41	0	3.22 to 2.20	0

* Submergence is based on the ratio of the upper and the lower heads.

FIG. 36.—DISCHARGE, 120 CUBIC FEET PER SECOND, IMPROVED VENTURI FLUME, LAS ANIMAS CONSOLIDATED CANAL.

FIG. 37.—DISCHARGE, 90 CUBIC FEET PER SECOND, IMPROVED VENTURI FLUME, LAS ANIMAS CONSOLIDATED CANAL.

fact that submergence is less, is probably the explanation of the increase in loss through the outlet section of the flume. It should be noted, also, that the loss of head in this case is measured from the upper end of the throat instead of the lower end.

Figs. 36 and 37 show the condition of flow during the tests made on the 10-ft. improved Venturi flume at Rocky Ford, Colo. Although the water surfaces appear equally smooth in the inlet section for both the 90 sec-ft. and the 120 sec-ft. discharges, the percentage loss of head is greater for the larger discharge, but not enough to be significant. The water surface in the outlet is much smoother for the 90 sec-ft. discharge than for the 120 sec-ft. discharge, but the percentage loss of head is greater. This is due to the fact that, as is shown in Table 5, the submergence is less in the case of the smaller discharge.

The loss of head through the improved Venturi flume, when the down-stream depth has been increased until it backs the water into the throat section and causes submerged flow, is shown in Table 6,* which is based on the submerged flow calibration data of the Venturi flume. These tests were made on the same flumes that were used in the free-flow tests and on similar discharges, and any change in the loss in head is due to the increase of the relative depth of water in the outlet end of the flume. The results show that both the inlet and outlet losses are materially less than the free-flow losses and also less than the losses observed by the author. They indicate that for low velocities at least, it is possible to obtain high efficiencies by proper control of the down-stream depth of water, even if the transitions are abrupt.

* Prepared from data of Irrigation Investigations of the Div. of Agricultural Eng., Bureau of Public Roads, U. S. Dept. of Agriculture, co-operating with the Colorado Agricultural Experiment Station.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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TRAFFIC CONTROL BY ELECTRIC SIGNAL LIGHTS

Discussion*

By Lewis W. McIntyre, M. Am. Soc. C. E.

Lewis W. McIntyre,† M. Am. Soc. C. E. (by letter).‡—The author discusses a phase of a problem involving the daily life of the vast majority of American people. Because the traffic problem effects almost 117 000 000, nearly every one feels that he has a solution for it. One man regards electric signals as a cure-all, another depreciates their value; one man wants to increase the speed limit, another wants to decrease it; one man favors elimination of parking, another wants to decrease fines for parking violations. Even partial solutions for a problem as complex as that of traffic, demand the powers of a keen analyst, and engineers would do well to give considerable attention to it. Electric traffic control has grown by leaps and bounds (roughly, from 1 000 signalized corners in 1924 to 8 000 in 1926), and as Mr. Eldridge states,§ it gives promise of becoming a dominating factor in the relief of traffic congestion.

The progressive or co-ordinated system has many advantages over the synchronized system, but engineers should not advocate the cut-and-try method of changing from one to the other. The writer has been informed that the system in Washington, D. C., was subject to considerable criticism and opposition during the period of this cut-and-try process. Signal settings based on careful engineering analysis of the traffic factors involved, can be made to operate with reasonable satisfaction immediately on installation. Traffic engineering work, at best, is subject to constant criticism and anything helping to minimize opposition is well worth while. Furthermore, many persons, some of them engineers, still believe that traffic control is entirely a police function and see no necessity for the engineer. Engineers should not encour-

* Discussion of the paper by M. O. Eldridge, Assoc. M. Am. Soc. C. E., continued from December, 1927, *Proceedings*.

† Asst. Prof., Civ. Engr., Univ. of Pittsburgh; Eng. Consultant, Pittsburgh, Pa.

‡ Received by the Secretary, January 23, 1927.

§ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1989.

age this viewpoint by suggestions that cut-and-try methods are satisfactory. In addition, the collection of necessary underlying data furnishes an engineering basis for varying the signal settings during special rush-hour periods and differing seasonal requirements, thereby fitting the system to the needs more accurately.

The need of short cycles in the operation of electric traffic signals cannot be too strongly emphasized. For maximum efficiency, pedestrians, as well as motor vehicles, must be governed by the signals; but many pedestrians will not remain on the sidewalk longer than 20 to 30 sec., particularly if cross-traffic at the moment is not very heavy.

In central business districts, the limited reservoir capacity of many existing streets furnishes another reason for a short signal cycle, particularly where blocks are short. In many cases, vehicles turning into a short block on the main streets from the cross streets, will entirely fill that block, if the cycle is long, before getting the signal to move. The resulting delay of these vehicles in getting started when the signal changes, will seriously delay traffic on the main street, even to the extent of tying up the system.

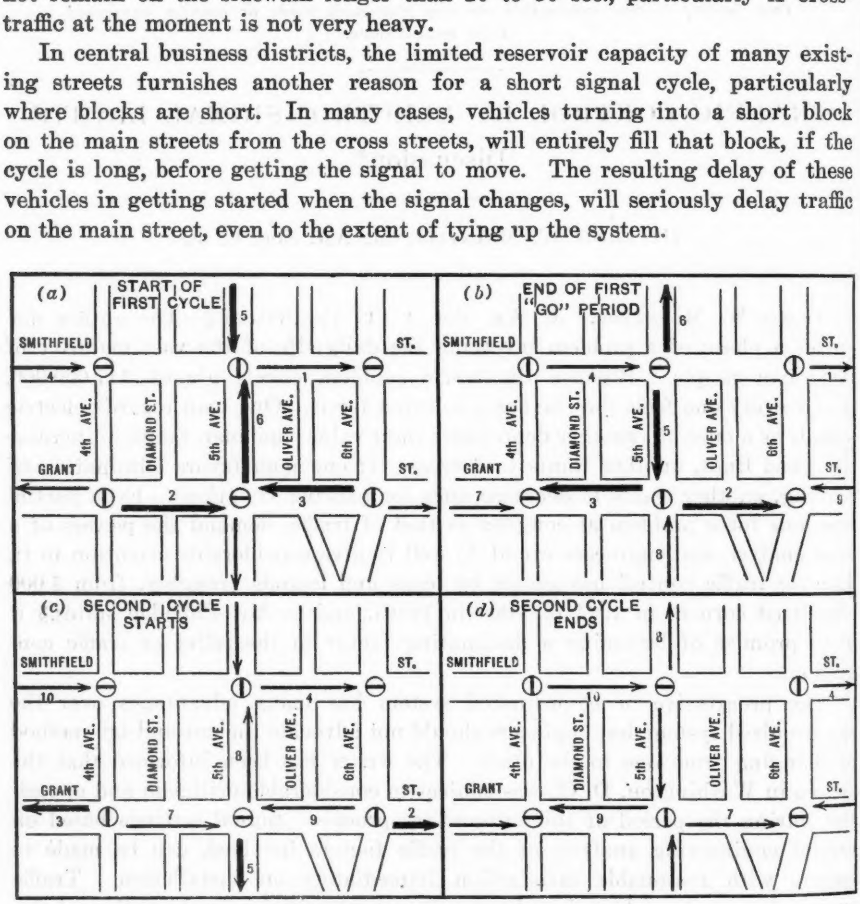

Fig. 4.—Disadvantages of Long Cycle.

Moreover, the use of long cycles with the co-ordinated system may result in the absolute stoppage of traffic in short blocks. A long cycle means a long group or platoon of motor vehicles which, if it is two blocks long, will completely tie up all cross traffic. Figs. 4 and 5 illustrate such a case that occurred recently in Pittsburgh, Pa., where a signal installation covering the entire central business district was designed. Some of the blocks were just a little

longer than 200 ft. Fig. 4 shows the situation as it would exist if a 70 to 80-sec. cycle were adopted for the signals on Grant Street.

At the beginning of the first cycle (Fig. 4(*a*)), Groups 2 and 3 on Grant Street are signaled "Go" at Fifth Avenue. Likewise, Groups 5 and 6 are signaled "Go" at Smithfield Street. Oliver Avenue and Diamond Street are blocked at Grant Street and Diamond Street is open at Smithfield Street.

When the signals change (Fig. 4(*b*)), Group 3 at Fourth Street and Group 2 at Sixth Street are ready to proceed along Grant Street, and on Fifth Avenue, Group 6 has just cleared Smithfield Street. Oliver Avenue and Diamond Street are still blocked at Grant Street and Oliver Street is open at Smithfield.

The corresponding periods of each succeeding cycle (Figs. 4(*c*) and 4(*d*)) cause the same conditions to be repeated in alternate order. In other words, traffic on Diamond Street and Oliver Avenue is prevented, by the effect of signals, from ever crossing Grant Street, and at Smithfield Street, it is free to cross only once in every other cycle.

With the same symbols, Fig. 5 shows the situation if the recommended cycle of 45 sec. is used. Traffic on Diamond Street and Oliver Avenue then has an opportunity to move in its turn.

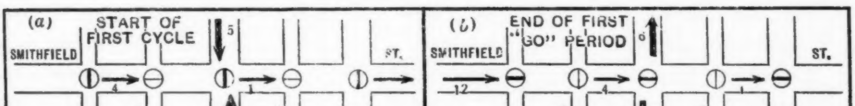

FIG. 5.—ADVANTAGES OF SHORT CYCLE.

However, with the short cycle, it is essential that the amber period, if used, be reduced to a minimum; otherwise, a too great proportion of the total time is set aside in which traffic cannot move. With a 40-sec. cycle and two 5-sec. amber periods, 25% of the available street time would be devoted to clearing the intersection, and only 75% to through traffic movement.

If the amber period after the red (a period serving no really useful purpose) is eliminated, this inefficiency is reduced one-half; but it can be still further reduced. Some timing studies conducted by the writer show that it required about 6 sec. for the average pedestrian to cross the street or reach a safety island. To be fair to the pedestrian, there should be, therefore, a 6-sec. amber signal period, so that if he has started to cross the street on the green signal (even if only during the last second of the green), he will have sufficient time to complete the crossing in safety and with at least a moderate amount of dignity.

For the vehicle, however, a warning of 2½ sec. is sufficient. Both purposes can be accomplished by showing the amber light for 6 sec., but allowing the first 3½ sec. to overlap the green, while the remaining 2½ sec. is made non-overlapping. Vehicles can then be permitted to enter the intersection as long as the green is showing, but not during the non-overlapping amber periods. Under these circumstances the total time set aside for a distinctly clear intersection period is reduced to only 2½ sec. This is only 6¼% of the total time cycle; yet all users of the street, including the much neglected pedestrian, are given ample warning of a change in signal.

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PAPERS AND DISCUSSIONS

TRAFFIC CONTROL IN NEW YORK, N. Y.

Discussion*

By Messrs. Russell S. Wise, C. D. Curtiss, and E. P. Goodrich.

Russell S. Wise,† Assoc. M. Am. Soc. C. E.—Mr. Hoyt has emphasized clearly the principles of traffic control in New York City from the viewpoint of the police. Apparently, they consider that the solution of the problem ends with the enforcement of regulations designed to keep traffic moving. Little or no consideration is given to the pedestrian who, under the two-color system, has the right to proceed across an intersection on the green only, while operators of motor vehicles are permitted to cut across his path in making right-hand turns. It would seem reasonable to give serious consideration to the use of a light for the pedestrian alone.

A three-color system with an amber or yellow light on which there would be no movement of vehicular traffic would seem to be the solution of the present-day problem of protecting the pedestrian. In such a three-color system the pedestrian would have the right to cross an intersection on the green, but would do so subject to the inconvenience of having to be on the lookout for turns on the part of vehicles, while, if he desired to be more cautious, he could wait for the amber or yellow light, when he alone would have the right to cross the intersection.

The trend toward uniform traffic rules and regulations is spreading throughout the country. New Jersey will have the recommendations of the New Jersey State Traffic Commission presented in its Legislature during the present (1927-28) session. In principle, all rules and regulations concerning traffic should be uniform, and the ideal of uniformity would be realized if there were uniformity, not only within individual States, but as among groups of neighboring States.

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* Discussion on the paper by Philip D. Hoyt, Esq., presented at the meeting of the Highway Division, New York, N. Y., January 20, 1927, and published in the October, 1927, *Proceedings*.

† Cons. Engr. (Wise & Watson); Chairman, New Jersey State Traffic Comm., Passaic, N. J.

No agency is better suited to help in creating this uniformity than the Society which, through its wide influence, could assist materially in bringing about the proper legislation for the purpose.

C. D. CURTISS,* M. AM. SOC. C. E.—Ten years ago (1917) it was not at all uncommon to see estimates of a so-called motor-vehicle saturation point. Most of them have already been exceeded, and such estimates are not now so frequently encountered. Mr. Hoyt has cited figures showing the very rapid increase in the number of motor vehicles in New York City and all those vehicles at some time or other are used in the streets.

At present, engineering energies are largely directed toward getting the greatest use of existing street facilities that is consistent with safety. At important intersections, however, traffic on one street or the other is continuously blocked. To relieve this condition, which congests traffic so greatly, it will undoubtedly become necessary in the not distant future to provide grade separations. The time is coming when grade crossings will be eliminated from the most heavily traveled rural highways. The problem of grade separation on city streets presents a more difficult problem, but the economic waste due to delayed traffic will ultimately force a remedy. Double or even triple decking of streets without cross interference on the important arteries may be the solution.

E. P. GOODRICH,† M. AM. SOC. C. E.—About 1912, while the speaker was working with the City of New York, both in the capacity of the Aldermanic Traffic Committee Chairman and the Consulting Engineer of the Borough of Manhattan, data were secured with reference to the time interval used by each traffic officer the whole length of Fifth Avenue, from 14th Street to 59th Street, all day long. Those data were plotted on a time schedule with the different avenues as different spaces across the strip. It made a diagram that looked more like a pianola record than anything else. It was the study of that record which led to the original suggestion of what was then called the "Platoon System".

In other words, by arranging those periods so that there was a progressive movement of the period along the Avenue and across the cross streets, it was possible to develop a plan that would permit all traffic to move continuously, either on a one-way avenue or a two-way avenue. It is that system, which has been developed lately in Chicago, Ill., and, to a certain extent, in Los Angeles, Calif., and in a great many other communities.

One of the difficulties in applying that system to-day has been suggested with reference to the co-ordination between vehicular traffic and street-car traffic. Wherever the law requires a stop behind a street car, there is obviously nothing that can be done. In those cities, however, where loading platforms are provided, and where a vehicle can move past a street car if there is a sufficient clearance, it is entirely possible to arrange the schedule so that the street car, as it moves, and as it stops for taking on passengers, will have the same periods of operation as the vehicles. In other words, the vehicles can

* Asst. to the Chf., U. S. Bureau of Public Roads, Washington, D. C.

† Cons. Engr., New York, N. Y.

move twice as fast as the street cars and still keep the latter moving continuously. The street cars can move at an average of 12 miles per hour, including stops, and the vehicles at 24 miles per hour without stops.

When it comes to arranging a diagonal thoroughfare in connection with the longitudinal system, by changes in speed (which obviously must be posted), it is also possible, as far as the use of diagrams can demonstrate it, to arrange a traffic signal system on this progressive or wave or platoon method, which will interlock everything. In fact, the speaker has a wooden model that shows how the time-space diagram can be fitted in with a cross-town interlocking system so as to make everything co-ordinate absolutely.

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PAPERS AND DISCUSSIONS

BAFFLE-PIER EXPERIMENTS ON MODELS OF PIT RIVER DAMS

Discussion*

BY MESSRS. ROBERT F. EWALD AND F. W. SCHEIDENHELM

ROBERT F. EWALD,† ASSOC. M. AM. SOC. C. E. (by letter).‡—In 1915 the writer obtained some data relative to baffle-piers at the toe of high over-fall dams. This information is presented herewith in the hope that it will prove to be a worth-while addition to the general store of knowledge on the subject to which the authors have contributed so much.

The data were obtained as part of a rather elaborate series of experiments, begun in 1914, on the hydraulic jump in relation to high over-fall dams. A brief series of experiments on baffle-piers was made in the following year, with the same model dams used during the previous year's work. The primary object of the second year's work was to determine the extent to which baffle-piers could be depended on to reduce the high velocities at the toes of high dams and thus to reduce the required depth of the hydraulic jump. The experiments were made on an ogee type of spillway built at the upper end of a rectangular wooden flume. At the lower end of this flume, at a distance of approximately six times the height of the dam, was an adjustable weir that was used to back up the water for forming the hydraulic jump as desired at the toe of the dam. With the weir removed, the velocity throughout the channel down stream from the dam in all cases was considerably greater than $\sqrt{g\,d}$ (in which, d is the depth of flow), so that the water was always flowing at the lower alternative stage. The depths of water required to develop the hydraulic jump and bring it to a predetermined position at the toe of the dam were determined by experiments with varying depths on the crest of the dam. The required depths thus found conformed very closely to a smooth curve as shown in Fig. 41.

* Discussion of the paper by I. C. Steele and R. A. Monroe, Members, Am. Soc. C. E., continued from March, 1928, *Proceedings*.

† Asst. Engr., The Aluminum Co. of America, Pittsburgh, Pa.

‡ Received by the Secretary, February 14, 1928.

The problem arose as to reducing the velocities sufficiently to form the jump and one of the suggestions was the use of baffle-piers. It seemed that if they possessed any pronounced efficiency in absorbing the high over-fall velocities, such efficiency could be determined by noting the reduced depth of water required to develop the hydraulic jump. In following out this idea, a line of baffle-piers was attached to the bucket of the dam, as shown in Fig. 42. These baffles were of the same height as the estimated maximum depth of water at the point where the velocity was greatest, and were staggered so that the deflected jets presumably would "interfere" with each other to the greatest possible degree.

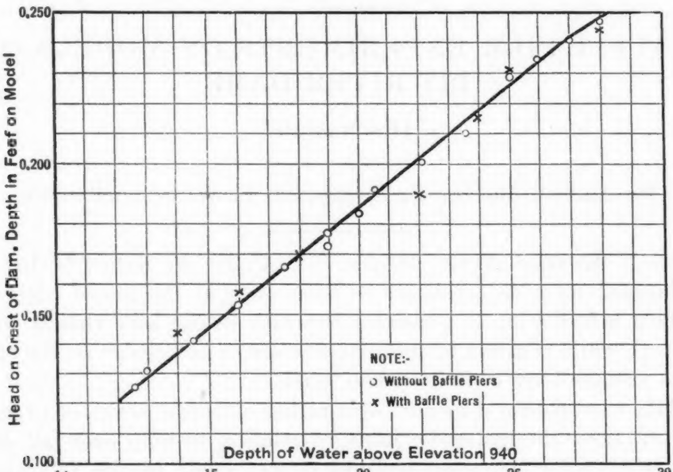

FIG. 41.—DEPTHS OF WATER REQUIRED TO BRING JUMP TO TOE OF DAM WITH AND WITHOUT BAFFLE-PIERS.

Another series of experiments was run, and, as before, the head on the crest of the dam was obtained as well as the depth of water (in the down-stream channel), that was required to bring the beginning of the jump to the predetermined position at the toe of the dam. It was a rather difficult matter to establish the beginning of the jump with any degree of accuracy and a somewhat indirect method was required. The procedure was to establish the jump clearly at the extreme down-stream end, then to build up the pool depth, watching the movement of the jump up-stream, until its beginning was lost in the disturbed flow around the piers. By relating the increment of movement to the increment in depth, it was possible to make a fair estimate of the depth required to bring the beginning of the required jump to position. These estimated depths were plotted on the curve, previously developed (see Fig. 41) and since they fell indiscriminately above and below, it appeared to the experimenters that the energy dispelled by the piers was relatively so small in proportion to that absorbed by the jump that further experiments along this line would not prove of particular profit. Comparative photographs were taken, similar to those shown in Figs. 43 and 44, to

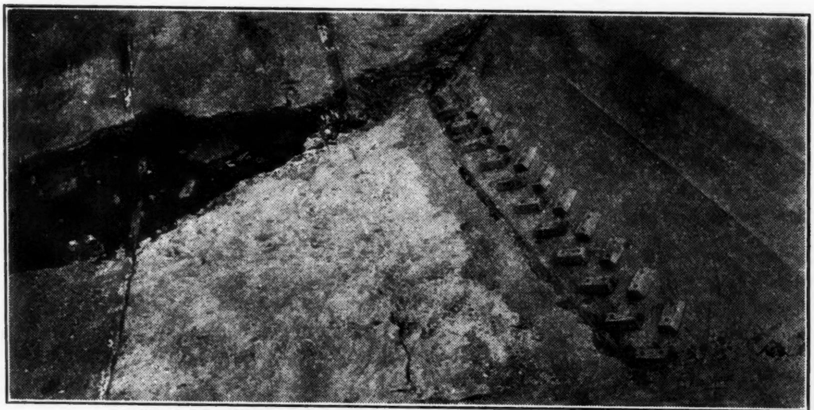

FIG. 42.—BAFFLE-PIERS ATTACHED TO BUCKET OF MODEL.

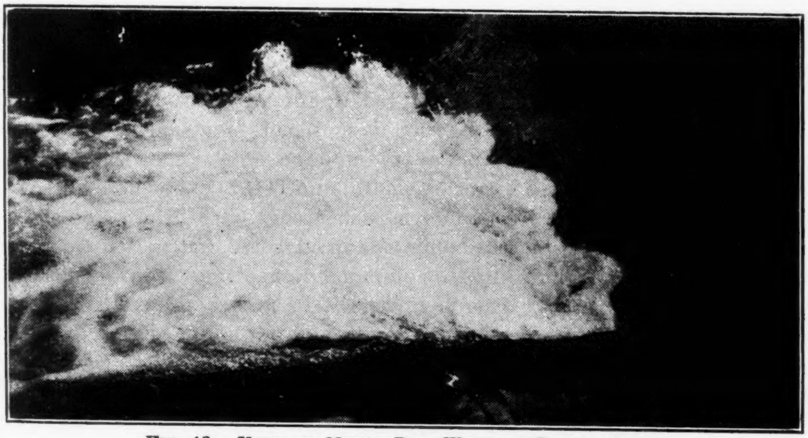

FIG. 43.—VIEW OF MODEL DAM WITHOUT BAFFLE-PIERS.

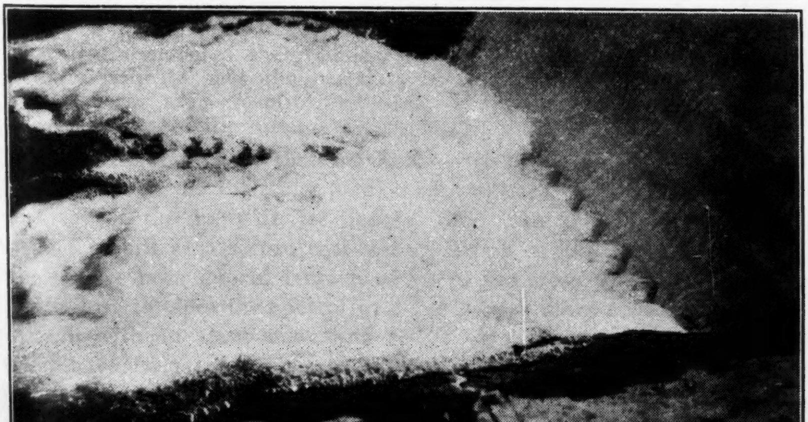

FIG. 44.—VIEW OF MODEL DAM WITH BAFFLE-PIERS.

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record conditions with and without the baffles. There was little question but that, with the baffles in, the surface action of the water was much smoother than with them out, but as the writer was primarily interested in the conditions at the bottom of the channel, which presumably were reflected by the net decrease in velocity of the water and freedom from its up-stream movement, it appeared that there would be little practical gain in the use of baffles. In fact, they seemed to possess a practical disadvantage in that they would be subjected to tremendous battering action and must be repaired from time to time. Furthermore, unless they were very skillfully designed and placed, and the gates on the dam correctly operated, water would be sprayed everywhere. This was a positive detriment with the power-house locations then under consideration.

It is apparent from Figs. 42, 43 and 44, that the internal interference caused by these baffles is less than that obtained in the Pit River experiments; yet having had the "first chance" at the high velocities, these baffles apparently should have caused a higher percentage of reduction in velocity than piers located much farther down stream and nearer the middle of the jump. Unfortunately, the writer developed such an unfavorable opinion of the baffles as an adjunct to the jump that he did not attempt any systematic experiments with them at locations farther down stream or with different shapes and positions. This opinion was somewhat affected by experiments made the previous year (1914) on suitable forms of secondary dams for developing cushion pools. One type consisted of heavy square-nosed piers spaced closely enough together to back up the water sufficiently to form the jump. It would have taken much more concrete to build piers of sufficient size for this purpose than for a dam of the straight ogee type. Having tried piers at both ends of the jump, the middle did not appear very promising and was completely neglected.

The principal requisites for good action in the hydraulic jump were clearly established during the experiments. These principles, with some additions, have been clearly expressed* once more by Sherman M. Woodward, M. Am. Soc. C. E. One of the requisites stated by Professor Woodward is:

"That if the slope of the jump off be made greater than one in three, experiments indicate that the certainty and effectiveness of the jump in destroying energy are perceptibly reduced. If the slope is made as steep as 45°, the usefulness of the jump is mostly lost".

It appears to the writer that the steep slope of the down-stream end of the bucket used on the Pit River dams must interfere very seriously with the development of the jump in its desired form. In the Pit River experiments, the inability to secure satisfactory action of the jump without the piers may have been due in large measure to the steep slope of the bucket and the failure to place the control weir far enough down stream. Dam No. 3 was built with an apron extending 150 ft. down stream from the bucket. It would have been interesting to know the results that would have been obtained by lowering the bucket until its lower edge was nearly tangent to

* In a letter to *Engineering News-Record*, December 15, 1927, entitled "Toe Erosion Below Overfall Dams."

the apron, and then placing a single line of piers about 100 ft. from the edge of the bucket instead of 47 ft. to the more distant pier. The baffles then would act as a control weir. The end results probably would have been nearly the same as those secured by the authors, but the baffle-piers would then have been saved from the tremendous battering to which they are now subject.

Any one who has watched and experimented with the hydraulic jump as developed under suitable conditions cannot avoid marveling at its efficiency in absorbing high velocities without effect on the channel, and the conviction grows that in its action lies the secret of the success of most over-fall dams, otherwise many structures standing to-day would have disappeared as the result of erosive action. Whether or not artificial construction is really necessary to develop the jump in its most efficient form, depends entirely on local conditions. Usually, there is no serious objection to permitting scour, providing the down-stream channel conditions and the design of the bucket are such as to prevent eddies scouring backward toward the toe of the dam. Eventually, Nature will scour a pool sufficiently deep to permit the development of the jump in its most efficient form and the scouring action will then cease.

Where scour in the immediate vicinity of the toe is undesirable, it appears entirely logical to construct a smooth apron in front of the dam just as was done at Pit River, carrying it far enough down stream so that the hole that will be scoured at the end of the apron will not be too close to the dam; then "letting it go at that". It is true that some of the apron may be lost in the hole that will be formed at its end; but it is equally certain that eventually a natural cushion pool will develop and further scouring will cease. It is only a question of determining the size of pool required and making the apron long enough so that, assuming the rock to have been cut backward under the apron, the final pool will be of the required dimensions and far enough away from the dam to avoid trouble. Of course, where eroded material may land in tail-races, such procedure is not permissible, and provision must be made for an artificial pool. Very high over-fall dams are generally founded on exceptionally good rock and long aprons usually are not necessary. The very hardest rock will be eroded, and at a surprisingly fast rate in many cases, but such erosive action through proper design of the bucket can be forced to take place at a considerable distance from the dam and dangerous "back lash" eliminated.

Just what constitutes "proper design" has not yet been clearly established, but that it is attainable is illustrated by two high dams located on Little Tennessee and Yadkin Rivers in North Carolina, designated as the Cheoah and Narrows Dams. At Cheoah Dam, in which the drop from the spillway crest to normal water level is 175 ft., the lip of the bucket is about 5 ft. below the normal water level, and there is a natural pool about 30 ft. deep at low water, the depth rapidly increasing with higher stages. This dam has had many floods over its crest with 19 ft. of water passing through some gates. Ever since the last and most severe flood in 1920, which caused considerable scour

at some distance from the dam, gravel and loose stone have actually been piled against the toe of the dam below the lip of the bucket, indicating complete absence of erosive eddies at the toe. At the Narrows Dam, the drop from the crest of the dam to normal water level is 170 ft.; the lip of the bucket is at normal tail-water level, and the stream in front of the dam is relatively shallow. Considerable scour has taken place much closer to the dam and in a few spots the rock has been taken out directly in front of the bucket for a depth of 6 ft. In no place has loose material remained against the toe of the dam below the lip of the bucket. The rock at the Narrows Dam is considerably more friable than that at the Cheoah Dam, but this does not explain why loose material will pile against the bucket at one dam and not at the other.

The differences in conditions must be explained by (1) differences in natural pool depth; (2) differences in position of lip of bucket (designated "jump off" by Professor Woodward) relative to bottom of pool; and (3) whether or not the water is permitted to flow directly away from the dam or is forced by obstructions into major eddies in the cushion pool. (By obstructions is meant large masses of rock or heavy structures projecting into the thread of the current.) Major eddies may also be caused by jets of water directed into the pool at large angles to the main flow down the dam. At the Cheoah Dam the natural pool is three times as deep as at the Narrows Dam, and the relative position of the lip of the bucket to the bottom of the pool is much lower than in the second. At Cheoah, it has been possible to turn water into the pool in a fairly uniform direction, and the pool is relatively free from major obstructions, whereas at Narrows some water has been turned into the pool at a fairly large angle to the main flow and rock obstructions also develop large eddies.

To the writer it appears that the ideal method of absorbing kinetic energy of water at over-fall dams is to take the water through the entire drop over perfectly smooth surfaces into cushion pools, functioning in accordance with the laws governing the formation of the jump, and avoiding all obstacles to the free and direct flow of the water away from the dam, with the exception of those primarily designed to increase the depth of water at the toe of the dam, providing they are far enough away from the dam to permit the formation of the jump. For economic reasons scouring usually can be permitted, care being taken to predetermine the probable ultimate dimension of the pool and to see that it is eroded at a reasonably safe distance from the dam. Ultimately, by the formation of the jump under ideal conditions, the erosive action will cease, and no further trouble may be expected so long as the bucket is maintained in its proper condition.

F. W. Scheidenhelm,* M. Am. Soc. C. E. (by letter).†—The authors' statements‡ and their description of the tests indicate that observation of the surface of the water and photographic study were largely relied upon as a

* Cons. Engr. (Mead & Scheidenhelm), New York, N. Y.

† Received by the Secretary, February 29, 1928.

‡ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2191.

basis for comparing the various experiments. The importance attached by the authors to surface appearance is further emphasized by their characterization* of the results of Experiment No. 13 which, in model form, approximated the final construction. However, the writer's experience in experimentation by means of models, as to energy destruction and scour at the foot of spillways of dams, indicates strongly that the appearance of the surface of the water flowing from the foot of the spillway is not a reliable criterion regarding the effect on the stream bed. After all, the most reliable criterion for comparison of the various means of preventing scour is the action on the stream bed itself.

Experiments made under the writer's direction at the hydraulic laboratory of Cornell University indicated that the relative height of tail-water is of great, even of controlling, importance. In the model dam tests for determining the design of Pit River Dam No. 3, there was provided a "back-water control", as shown in Figs. 2, 3, and 4.† Apparently, this control or block (perhaps of wood in the model), was not embodied in the final or full-sized structure (see Fig. 1‡). However, the writer's experiments indicate that the omission, in the full-sized structure, of a control located so close to the foot of the spillway would result in a performance differing from that in the model. In particular, he suspects that there would result somewhat greater scour of the stream bed than if the equivalent of such a block or control were embodied in the final construction.

In the case of the model experiments for the dams of lesser height (No 4 Dam)§ a so-called "two-level" apron was tested and a similar apron was embodied in the final construction. The writer is inclined to the belief that the "back-water control" (again, apparently, a block), in the model would be of greater significance in the case of the higher apron, which is shown at the far side of the model in Figs. 17,‖ 26, and 27.¶ The near, or lower, side of the apron would seem, in effect, to involve a stilling-pool, terminating in the equivalent of a solid transverse baffle-wall nearly 14 ft. high and located about 70 ft. down stream from the vertical face of the spillway bucket (both dimensions are in terms of the full-sized structure). The writer has not overlooked the authors' statement** that "the difference in the final result, as indicated by the velocity in the river channel below, was, as nearly as could be judged, uniform across the width." However, here, too, one may infer that judgment was based primarily on surface appearance, which may be misleading. Moreover, even the authors state** that "the 'killing' of velocity was accomplished with less commotion on the deep side than on the shallow."

A study of Figs. 26 and 27 indicates that the right or far bucket was swept clear, whereas the left or near bucket, namely, that opposite the equivalent of a "stilling pool," was covered by back-roll. This is, to some

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2201.
† *Loc. cit.*, pp. 2192–2193.
‡ *Loc. cit.*, p. 2191.
§ *Loc. cit.*, p. 2201.
‖ *Loc. cit.*, p. 2205.
¶ *Loc. cit.*, p. 2215.
** *Loc. cit.*, p. 2218.

extent, a measure of the effectiveness of a given arrangement for the prevention of scour.

Inasmuch as the experiments described by the authors apparently did not involve simulation of the stream bed, nor any observation of scour, it is the more desirable and important that the results on the stream bed should be observed at the foot of the actual structures.

The writer is very sympathetic with regard to the difficulties involved in experimentation of this kind and thoroughly appreciative of the work of the authors, both in the experimentation itself and in their publication of the results. He hopes that they will share with their fellow members of the profession the results of the two full-sized constructions, especially as regards scour under flood conditions, just as they have in this paper, so freely and well, shared the results of the experimentation.

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PAPERS AND DISCUSSIONS

MAXIMUM FLOOD DISCHARGE IN SAN JOAQUIN VALLEY, CALIFORNIA

Discussion*

By Charles H. Lee, M. Am. Soc. C. E.

Charles H. Lee,† M. Am. Soc. C. E. (by letter).‡—This paper describes a method of estimating maximum flood discharge which assumes that within the same meteorological province the records of several streams, similar in storage, vegetation, soil, and other physical characteristics, can be combined, after making certain adjustments, so as to give an equivalent record for one of the streams. The adjustments include corrections for geographic position, average elevation and size of drainage area, and relation of peak to 24-hour flood discharge. The author states§ that he adopts this method because of the inherent limitations of flood discharge formulas and the impracticability of extending their use to streams other than those for which they were originally devised.

The writer's experience in estimating flood discharges from formulas confirms these conclusions, but has led him to favor the rational method rather than the stream-group method. Drainage areas have individual characteristics. These are much more effective in producing differences in flood discharge than they are in the case of total run-off. With certain modifications, the author's method might be applied to the eleven streams which he studied, if his purpose was to determine long-time, mean total run-off, but it is not logically applicable to the problem of determining flood discharge.

Throughout the paper the writer detects a confusion of thought regarding factors which influence total run-off and maximum flood run-off. It is believed that a rational analysis of these factors may be helpful.

Total Run-Off.—The total run-off from a drainage area is usually considered for an annual cycle (or water-year) and depends on the total precipi-

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* Discussion of the paper by Oren Reed, Assoc. M. Am. Soc. C. E., continued from February, 1928, *Proceedings*.

† Cons. Hydr. Engr., San Francisco, Calif.

‡ Received by the Secretary, February 3, 1928.

§ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2220.

tation on the drainage area during the period, minus losses due to evaporation from water, snow, ice, and soil, and transpiration from plants. Cyclic surface, snow, or ground storage may be important if wide variation occurs in precipitation from year to year. Total run-off may be expressed for unit area by the formula, $r = P - L \pm S$, in which, r is the total run-off from a unit area; P, the depth of annual precipitation; L, the depth of annual losses; and S, the annual storage correction.

Water, snow, and ice evaporation depend on the extent and period of exposure of the respective surfaces, and the temperature, humidity, and other factors which influence the evaporation rate. Elevation and geographic location as they affect meteorological conditions play an important part in fixing these losses. Soil evaporation depends on slope, quantity, and time distribution of rainfall, vegetation, depth, and porosity of the soil, and the factors which influence the rate of water evaporation. Transpiration depends on the type and extent of vegetation, which, in turn, depends on soil, rainfall, and elevation and geographic location as they affect meteorological conditions. Cyclic storage ordinarily becomes important only when artificial surface storage of large capacity has been constructed, or large ground storage capacity exists in the form of gravel beds or volcanic débris.

Flood Discharge.—In contrast to total run-off, this factor depends on the intensity of rainfall and the amount of run-off finding its way immediately into drainage channels. It may be expressed for a unit area by the formula, $q = Ci$, in which, q is the discharge, in second-feet, for a unit area; i, the intensity of rainfall, in inches per hour; and C, the run-off coefficient representing the ratio of the rate of immediate run-off to rainfall intensity and expressing the physical characteristics of the drainage area.

As the automatic rainfall records obtained during the past 20 to 30 years have demonstrated, rainfall intensity at any point varies greatly with time, being greatest for periods of a few minutes, decreasing rapidly for longer periods, and approaching a more constant value for periods exceeding 12 hours. Maximum flow at any point on the stream results from a rainfall the duration of which equals or exceeds the time of concentration of water from the most remote point (in time) on the drainage area. It is greatest when the surface is frozen or saturated from a previous storm and may be augmented in special cases by the effect of warm rain in melting snow on the ground.

The maximum rainfall intensity to be considered for a given drainage area is not uniform within the same "meteorological province", but varies from place to place depending on local conditions and especially on the time of concentration. The latter differs for every point on the stream. It may also differ at the same point for different storms, depending on the extent of the area upon which precipitation falls in the form of rain. For example, precipitation might fall in the form of snow on the higher portion of the drainage area, or the maximum storm might not extend over the whole of the area.

The time of concentration is controlled by the following factors:

1.—The time of flow over the surface of the ground to the nearest stream channel. This is influenced by the average slope of the

ground at the most distant point in the drainage area and the irregularities of the surface produced by vegetation and character and condition of the soil surface.

2.—The time of flow in stream channels. This is determined by the average slope and character of channel, as reflected in the value of n in Kutter's formula.

3.—The shape of the drainage area, whether long and narrow, fan-shaped, etc.

4.—The arrangement of branches and laterals.

5.—Surface storage.

The percentage of rainfall which finds its way immediately into the stream depends on:

(*a*) Losses, consisting of immediate evaporation (very small) and absorption. The latter is controlled by surface conditions, such as the extent of bare rock or frozen ground, the character of the soil, the slope, and the existence of a mat of vegetable material and humus.

(*b*) Temporary retention due to storage in irregularities of the surface and capillary action, the vegetation mat being an important factor in the matter.

(*c*) The degree of previous saturation, depending on the season of the year when the storm occurs and the relation of the time of the maximum storm to the storm period.

Table 8 summarizes these factors and serves to contrast the differences between those entering into total run-off and maximum flood run-off.

TABLE 8.—COMPARISON OF PRINCIPAL FACTORS CONTROLLING TOTAL RUN-OFF AND MAXIMUM FLOOD RUN-OFF FOR A GIVEN DRAINAGE BASIN OF AREA A.

Total run-off, $R = (P - L \pm S)\,A$.	Maximum flood run-off, $Q = C\,i\,A$.
A = superficial area of whole drainage basin.	A = portion of superficial drainage area covered by storm yielding rainfall of maximum intensity.
P = total depth of precipitation during complete run-off cycle (water-year).	i = maximum rate of rainfall of uniform intensity continuing throughout the period of concentration, or for the period of the storm if shorter than the period of concentration.
$L \pm S$ = depth of accumulated losses during the entire water-year by evaporation from water, snow, ice, and soil, and transpiration augmented or diminished by cyclic surface or ground storage.	$1 - C$ = percentage of rainfall at maximum rate lost by absorption and temporary storage in surface irregularities and capillary films.
	C = percentage of rainfall at maximum rate which runs off immediately and appears in the stream by the end of the period of concentration.

In the light of this analysis (see Table 8), the following conclusions are believed to be sound:

First.—The author's implied assumption* that the rainfall intensity producing maximum flood discharge is uniform throughout a meteorological province, or for all points on a drainage system, is not justified.

Second.—Few, if any, drainage areas (including their drainage systems) are so similar that both the times of concentration and the run-off factors

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2219.

for maximum discharge correspond, as the author's second assumption* would imply.

Third.—The items selected by the author for making adjustments between various drainage areas have no direct or apparent relation to the factors which control maximum flood run-off.

In comment on the third conclusion, the distance north or south of Sanger, Calif., has not been shown to be systematically related to the form of the rainfall intensity curve at different points, nor to any factor influencing flood discharge. Also, the average elevation of the drainage area, although indicating the percentage of precipitation falling as snow, is not so expressive of this relation as would be the proportion of the drainage area above the critical elevation. Fig. 8† shows no definite relation between flood run-off and average elevation. Then, again, the size of the drainage area, although it is an important factor for extremely large areas, or in regions of local storms where a given storm covers only a portion of the area, has little bearing on flood discharge from Sierra Nevada streams. The latter are normally visited by general storms the extent of which far exceeds the limits of the drainage area. Fig. 9‡ shows clearly the absence of any relation between drainage area and maximum flood flow for these streams.

The apparent conclusion illustrated in Table 3§ is that the peak flood discharge to be expected on Kings River at Sanger is 62 240 sec-ft. The maximum 24-hour flood reported for this station by the U. S. Geological Survey is 43 930 sec-ft. on January 7, 1901. Correcting this to peak flood by use of Fig. 10‡, a value of 73 800 sec-ft. is obtained, which exceeds the author's value by 18 per cent. Table 9 is obtained by assembling the various values given in the paper. It appears from this table that the author's value for peak flood on Kings River, at Sanger, is from 12 to 19% less than that derived by any other method. It is believed that the author's paper is incomplete without a discussion of the reasons which underlie the assumptions on which his method is based. This is especially true in view of the fact that it departs somewhat from accepted practice.

The rational method of estimating flood discharge was originally used in estimating maximum run-off from urban and suburban areas in connection with storm-sewer design. The same method is applicable to natural drainage areas of sizes exceeding those considered by the author, although for very large areas its use is impracticable because of the complexity.

The general application of the rational method is becoming more and more feasible as the appropriate basic data are accumulated. Automatic tipping-bucket rainfall records kept by the U. S. Weather Bureau and other public agencies, are now available at many points throughout the United States. At many of these stations records covering from 15 to 25 years are available for 5-min. periods, and up to 50 years for periods of 12, 24, 36, and 48 hours, etc. From such data rainfall intensity curves can be constructed which indicate the range of rainfall rates for various periods of time and frequency of occur-

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2219.

† *Loc. cit.*, p. 2234.

‡ *Loc. cit.*, p. 2235.

§ *Loc. cit.*, p. 2237.

rence. The normal expected intensities of rainfall at Oakland, Calif., within 2, 5, 10, and 15-year periods are as shown in Fig. 17. These curves are based on automatic recording rain-gauge observations covering 18 complete years of a 23-year record (1904 to 1927) at the Chabot Observatory. They are typical of the San Francisco Bay region. The same storms upon which these curves are based, extend over the western slope of the Sierra Nevada Mountains, including the drainage areas of the San Joaquin Valley streams considered by the author. In the absence of automatic rainfall records on the drainage area of Kings River and adjacent streams, the writer believes that this curve could be modified and extended from local foothill and mountain records for 12, 24, 36, and 48-hour periods so as to give a generally representative curve for this region, which would include all times of concentration that would occur in practice.

TABLE 9.—COMPARISON OF VARIOUS VALUES FOR PEAK FLOOD DISCHARGE OF KINGS RIVER, AT SANGER, CALIFORNIA.

Authority.	24-hour maximum flood, in second-feet.	Peak flood, in second-feet.
Modified Myers formula	......	77 500
Fuller formula (100 years)	......	71 300
Plate LXVIII, *Bulletin No. 5*, State of California, Department of Public Works (100 years)	41 000	70 520
Estimated flow, January 7, 1901, *Water Supply Paper 299*, U. S. Geological Survey	43 930	73 800*
Author (Table 3)	31 215	62 240

* Obtained from 24-hour flow by use of Fig. 10.

The time of concentration at any point on a stream is readily estimated by observing the movement of flood waves down the channel, checked by velocity measurements during high water. It is to be recognized, however, that the water velocity usually indicates a more rapid rate of advance than the flood wave because the latter is modified by channel storage. The determination is most effectively made in conjunction with rainfall observations, but, on a long stream course, the observed rates of travel with proper modifications may be extended back to stream head-waters. Time should also be allowed for the flow of water over the surface of the ground and down small tributaries before it reaches the larger channels.

The writer recently had an opportunity to determine the time of concentration for Strawberry Creek, a small stream flowing through the campus of the University of California at Berkeley and draining a canyon cut back into the hills on the east shore of San Francisco Bay. The area drained at the measuring weir is less than 1 sq. mile (509 acres) and has a range in elevation of from 1 200 to 1 500 ft. on the surrounding ridges to 500 ft. at the weir. The average distance from the ridge crests to definite stream channels is 0.25 mile with a difference in elevation of from 400 to 600 ft. The greatest length of stream channel is 1.2 miles with an average slope of 140 ft. per 1 000. The entire area supports a good growth of natural grass, with close-growing brush on the northern slopes and planting of young pines along the ridges at the head of the canyon.

Self-recording weir measurements of the flow were made by the University under the direction of H. B. Foster, Assoc. M. Am. Soc. C. E., during the five seasons between 1910 and 1915. Automatically recorded rainfall records from a Friez tipping-bucket rain-gauge on the University Campus were also kept during the period by the College of Civil Engineering, under the direction of Charles Gilman Hyde, M. Am. Soc. C. E. During the season of 1914-15, comparative observations of rainfall on the University Campus and at thirteen rain-gauges placed at critical points on the Strawberry Creek drainage area indicated a practical agreement between the rainfall as measured at the campus and the average falling on the drainage area. From a study of the original record sheets it was found that although both records were intermittent there were ten storms for which both were complete.

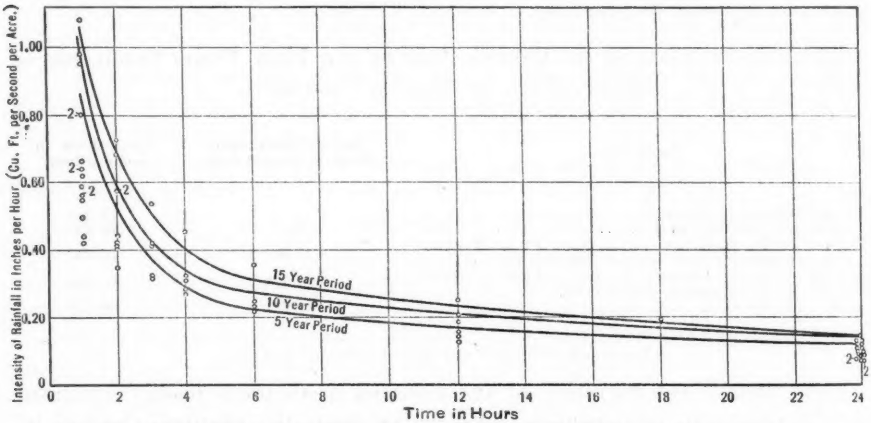

FIG. 17.—RELATION OF DURATION AND INTENSITY OF RAINFALL AT OAKLAND, CALIF.

The times of concentration for each storm were obtained by comparing the automatic record sheets for rainfall intensity and stream discharge (see Table 10). Observed maximum discharge for the stream exceeds that for any of the storms recorded, but plotting maximum discharge for each storm against the concentration period for each storm (data in Table 10), the time of concentration for maximum flood discharge at the weir was concluded to be 40 min. For large areas the time of concentration may be a matter of hours or even days.

The ratio of the maximum rate of flood discharge to the average maximum rainfall intensity which produces it has not been investigated to any great extent for natural drainage areas. Many observations have been made on areas more or less covered by urban improvements, in connection with storm-sewer design; but run-off factors for such conditions do not apply. The factors generally used on such work for previous surfaces, such as parks, gardens, lawns, and meadows, vary from 0.05 to 0.25, depending on surface slope and character of subsoil.

Merrill M. Bernard, M. Am. Soc. C. E., states* that values for watersheds will vary from 0.11 to 0.15, but does not describe the types of drainage

* *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1087.

area to which such factors apply. It is probable that he has in mind the milder climates and flat slopes. Other authorities quote values of from 0.50 to 0.75, probably for colder regions and steeper slopes.* Mr. George Chamier gives the following values:†

Frozen ground	0.66
Naked unfissured mountains, Very steep ground, Paved streets	0.80
Mountainous and rocky country or non-absorbent surfaces	0.55 to 0.65
Wooded hill slopes and compact or stony ground..	0.45 to 0.55
Meadows and gentle declivities, absorbent ground..	0.35 to 0.45
Flat country, sandy soil, or cultivated land.......	0.25 to 0.35

TABLE 10.—RATIO OF MAXIMUM RATE OF FLOOD DISCHARGE TO MAXIMUM INTENSITY OF RAINFALL DURING CONCENTRATION PERIOD FOR STRAWBERRY CREEK, BERKELEY, CALIFORNIA.

Storm number.	Date of storm.	Maximum Rate of Rainfall:		Maximum Rate of Run-Off, in Second-Feet per Acre.			Maximum discharge, in gallons per hour.	Concentration period, in minutes.	Run-Off Ratio for.	
		Time.	Increase per hour.	Time.	Total.	Increase due to storm.			Total run-off.	Increase due to storm.
1	1/13/11	10:00-12:00 A.M.	0.070	1:30 P.M.	0.004	0.002	55 000	90	0.06*	0.04*
2	1/13/11	4:00- 5:00 P.M.	.....	6:00 P.M.	0.016	0.014	220 000	60	....	....
3	1/13/11	9:45-10:00 P.M.	0.178	11:00 P.M.	0.021	0.016	290 000	60	0.12	0.09
4	1/19/11	8:15-10:30 P.M.	0.200	10:00 P.M. 11:00 P.M.	0.005	0.003	120 000	105	0.02*	0.01*
5	1/28/11	2:00- 4:00 A.M.	0.178	3:00 A.M. 5:00 A.M.	0.021	0.020	285 000	60	0.12	0.11
6	3/ 1/11	2:30- 3:45 A.M.	0.110	4:30 A.M.	0.015	0.013	205 000	60	0.13	0.12
7	12/16/12	6:30- 7:00 A.M.	0.073	8:30 A.M.	0.001	0.001	20 000	90	0.02*	0.02*
8	1/18/13	6:15- 7:00 A.M.	.....	9:30 A.M.	0.009	0.007	120 000	150	....	....
9	1/12/14	10:30-11:00 A.M.	0.335	11:45 A.M.	0.051	0.039	700 000	45	0.15	0.12
10	1/12/14	12:30- 1:15 P.M.	0.307	2:00 P.M.	0.068	0.039	930 000	45	0.22	0.12

* Ground dry at beginning of storm.

The Strawberry Creek records enabled the writer to determine run-off factors for a region of mild temperature and steep topography covered with grass and close growing brush. The percentage of water running off during the peak of the flood was found to be relatively small and the stream was frequently still swollen with delayed run-off from one period of maximum rainfall when another would occur. It was desirable to segregate the flow due to the current storm from that derived from previous storms. This was readily accomplished graphically upon the hydrographs by extending the slope of the diminishing flood flow to a point beneath the succeeding peak. The results for the ten storms are summarized in Table 10, which indicate that for major storms (January

* "Public Water Supplies," by Turneaure and Russell, Third Edition, p. 74.

† *Minutes of Proceedings*, Inst. C. E., Vol. CXXXIV, 1897–98, IV, p. 313.

12, 1914) occurring when the ground was still wet from previous storms, the ratio of maximum flood discharge to the rate of rainfall that produces it was 0.12. If delayed run-off from previous storms was added, the ratio became greater and reached 0.22 in one instance (afternoon, January 12, 1914). If it be assumed that the run-off from the preceding storm is still at peak flow when the peak from the succeeding storm arrives, the run-off ratio (in terms of rainfall) for the record storm might be 0.24, which represents a limiting value. For maximum conditions, a value of 0.25 was assumed.

It is thus apparent that the flood discharge run-off factor has a wide variation, depending on local conditions. This element in the application of the rational method is, at present, the weakest link. The writer regards the method as having real merit and believes that attention should be given by the profession to perfecting its application.

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PAPERS AND DISCUSSIONS

THE SCIENCE OF FOUNDATIONS—ITS PRESENT AND FUTURE

Discussion*

By Messrs. W. S. Housel and Frank S. Bailey

W. S. Housel,† Jun. Am. Soc. C. E. (by letter).‡—Without doubt, it is a source of satisfaction to many to see a revival of interest in the problem of the bearing value of soils. For the last few years there have been a number of men devoting a great deal of time and effort to a better understanding of foundation problems from the standpoint of soil conditions. Certainly, Professor Terzaghi's work has been outstanding in this respect, and he has made a valuable contribution to the science of foundations.

During the past year, the writer, acting for the Board of Wayne County Road Commissioners at Detroit, Mich., has been engaged in conducting a most extensive series of tests to determine the bearing value of soils. The work has been done in the field, under conditions governing actual construction, and, at the same time, every effort has been made to keep the accuracy of the data comparable to what might be obtained with a similar set-up in the laboratory.

More than fifty tests have been made on different sizes and shapes of bearing areas, some tests on piles and some combinations of bearing plates and piles. Measurements of lateral pressure and a study of the general phenomena of the transmission of pressure in soils constitute an important part of the investigation. The laboratory work is being conducted by the Civil Engineering Department of the University of Michigan. Through the courtesy of the Michigan State Highway Department, the facilities of the State Highway Laboratory have been made available for this work. With the field results of this investigation at hand a few comments on the principles of soil action as outlined by Professor Terzaghi may be of interest.

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* Discussion of the paper by Charles Terzaghi, M. Am. Soc. C. E., continued from March, 1928, *Proceedings*.

† Instr. in Civ. Eng., Univ. of Michigan, Ann Arbor, Mich.

‡ Received by the Secretary, February 16, 1928.

The most consistent results of all the tests made by the writer indicate that there are two factors of strength by virtue of which cohesive soils, such as clay, are capable of supporting loads. One of these factors is the shearing resistance on the perimeter of the bearing area; the other is the resistance of the soil to compression, due to a condition of strain set up in the soil around the bearing area. This last phenomenon has been called a pressure bulb.

During the progress of a test it is noticeable that the first failure taking place is failure in shear along the perimeter of the bearing area. In the latter part of the test there is a noticeable upheaval of soil surrounding the bearing area which indicates that a condition of strain exists. That this upheaval cannot be classified merely as flow of soil from beneath the bearing plate is shown by the fact that for some time after the upheaval is first noted the bearing plate continues to support the increased load applied with relatively small settlements. This action continues with increased loads up to a rather definite point of failure, at which point the soil flows from beneath the bearing plate with apparently no restraint. This marks the point at which the pressure bulb fails.

The load deflection diagrams obtained by the different tests bear out these general observations. Fig. 21 shows a set of typical load deflection diagrams, in this case for three different bearing plates, each with an area of 4 sq. ft. As shown, one plate was round, one square, and the third rectangular, with a ratio of width to length of 1 to 7. The round plate has a perimeter-area ratio, $\frac{P}{A}$, of 1.78; the square plate, of 2.0; and the rectangular plate, of 3.04. The material is stiff yellow clay and the size of the test pit is the same as the bearing plate, 4 sq. ft.

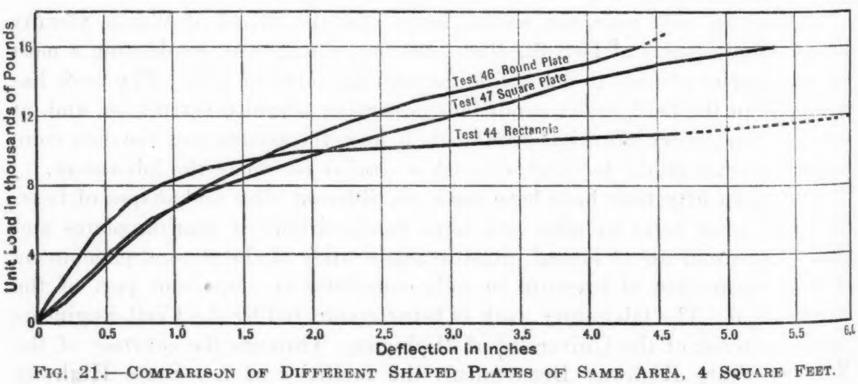

FIG. 21.—COMPARISON OF DIFFERENT SHAPED PLATES OF SAME AREA, 4 SQUARE FEET.

It is significant that in the first part of the tests, the range of bearing capacity in which shear failure about the perimeter is taking place, the relation between the bearing capacity of the several plates is similar to the relation between the values of x for the different plates. The bearing capacities are not in exact proportion to x because, even in their lower ranges, the pressure bulb is beginning to have some effect as a factor of strength.

In the latter part of the test, in the range of bearing capacities where the pressure bulb is a controlling factor, there is a complete reversal of bearing capacity relations. The plates with the largest $\frac{P}{A}$-values have the smallest bearing capacities. The important consideration in upper range of capacities is the effectiveness with which the different shaped plates adjust themselves to the pressure bulb. The strained condition in the soil, giving rise to the phenomenon of the pressure bulb, tends to become circular. Thus, the round plate is the best shape to take advantage of this factor of strength, the square plate is not quite so good, and the rectangular plate is the least effective of all.

Fig. 22 shows the load deflection diagrams obtained by testing areas of 1, 2, 4, and 9 sq. ft. in the same shape of plate. As in Fig. 21, the material is yellow clay and the pit is the same shape and size as the bearing plate in each case. The decreased capacity of the larger plates with smaller values of $\frac{P}{A}$ is the first impression gained from a study of these diagrams. Again, it is significant that the variation in bearing capacity is not in direct proportion to the perimeter-area ratios of the different plates. In general, it has been found that the larger the bearing area with its smaller perimeter-area ratio, the greater the discrepancy shown between the $\frac{P}{A}$ ratio and the bearing capacity.

On the first studies of the data obtained, it was thought that the perimeter-area ratio was the only variable affecting the bearing capacity. This is the natural result of testing small bearing areas for which $\frac{P}{A}$ is large and the variation in the ratios of the different plates is great. When the largest area tested was 9 sq. ft., the discrepancy between the bearing capacity and the corresponding perimeter-area ratio was not large. It is only when the principle is applied to the much larger areas used in practice that the fallacy of this assumption becomes apparent.

However, for the purpose of discussion, it may be profitable to start with this assumption, which is admitted to be false, and develop the principle as applied to the action of soils and see where it leads. It is assumed that the bearing capacity of two spread foundations of different sizes is proportional to their perimeter-area ratios, the settlement being constant in both cases.

Let p be the bearing capacity, P, the perimeter, and A, the area of the footings considered. Then for a constant settlement in the two footings, and according to the assumption.

$$\frac{p_1}{p_2} = \frac{P_1}{A_1} \div \frac{P_2}{A_2} \qquad \text{(7)}$$

If the bearing areas considered are round: $P_1 = \pi D_1$; $P_2 = \pi D_2$; $A = \frac{\pi D_1^2}{4}$; and $A_2 = \frac{\pi D_2^2}{4}$. Substituting these values in Equation (7) and simplifying:

$$\frac{p_1}{p_2} = \frac{D_2}{D_1} \qquad \text{(8)}$$

If the bearing areas are square: $P_1 = 4\,D_1$; $P_2 = 4\,D_2$; $A_1 = D_1^2$; and $A_2 = D_2^2$. Again, substituting in Equation (7) and simplifying:

$$\frac{p_1}{p_2} = \frac{D_2}{D_1} \qquad (9)$$

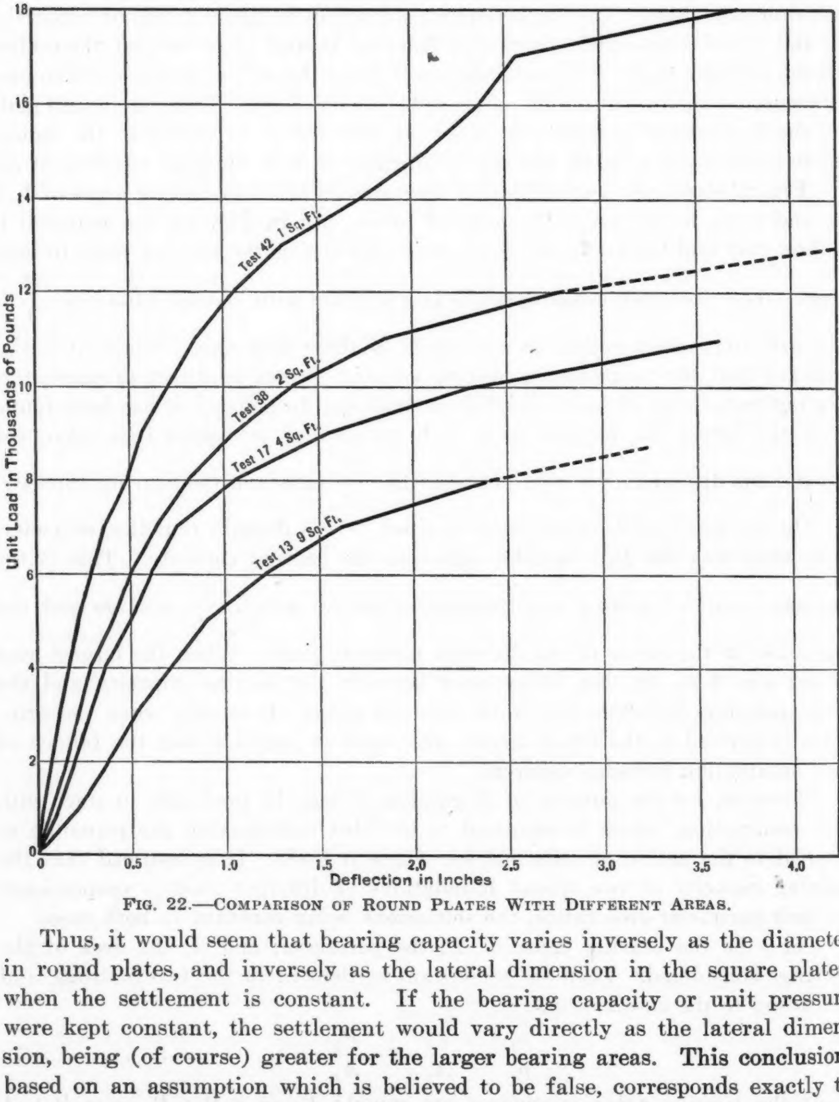

FIG. 22.—COMPARISON OF ROUND PLATES WITH DIFFERENT AREAS.

Thus, it would seem that bearing capacity varies inversely as the diameter in round plates, and inversely as the lateral dimension in the square plates, when the settlement is constant. If the bearing capacity or unit pressure were kept constant, the settlement would vary directly as the lateral dimension, being (of course) greater for the larger bearing areas. This conclusion, based on an assumption which is believed to be false, corresponds exactly to the theory as stated by Professor Terzaghi. As has been shown, this mistaken theory is the natural result of insufficient data on small sizes of test areas.

Aside from the consideration that the theory is based on a false assumption, there seems to be additional objection to the use of such data as have been obtained, in terms of settlement for different sizes of foundations. To state the theory in terms of bearing capacity for a given settlement seems to

be more direct and better adapted to application to practical problems than to leave it in terms of settlement for a given unit pressure. In addition, if the variation in bearing capacity were expressed in terms of $\frac{P}{A}$ instead of being further reduced to terms of lateral dimension, the theory would be more readily applicable to all shapes of footings. In the case of a rectangular footing, one is immediately confronted with the question of which lateral dimension should be used in reducing bearing capacity for different sized footings. If left in terms of the perimeter-area ratio, the rectangular footings, or any other shape, present no serious difficulty in dealing with the reduction of bearing capacity.

The primary object of this discussion is to point out that the theory which states that bearing capacity varies inversely as the lateral dimension of the footing, settlement being constant, or the theory which states that settlement varies directly as the lateral dimension, unit pressure being constant, is similar to the theory that the perimeter-area ratio is the only factor governing bearing capacity. Such a principle takes into consideration only one factor of strength, which is shear on the perimeter, and neglects the strength of the soil in compression which, in most cases, is the most important factor.

The fallacy of this theory of soil action becomes apparent when applied to large spread foundations, such as the case of the building in San Francisco, Calif., cited by Professor Terzaghi.* The data from tests for bearing capacity showed an average settlement of 0.10 in. for a unit pressure of 4 800 lb. per sq. ft. These tests were evidently made on a test area of 1 sq. ft. The building was placed on a mat foundation, 218 by 252 ft., designed to carry the dead load of the building at a unit pressure of 4 800 lb. per sq. ft., which was considered allowable.

Applying the proposition that settlement varies directly as the lateral dimension, unit pressure being constant, the settlement should amount to $252 \times 0.10 = 25.2$ in., or $218 \times 0.10 = 21.8$ in., depending on which lateral dimension is considered. The obvious absurdity of these results is excused on the basis that the soil is only slightly cohesive. The test data are quite representative of what might be obtained in a cohesive soil, and to expect a settlement under such conditions comparable to the results obtained from the equations given, would be no less absurd, even if the soil were of the most cohesive type.

To analyze the test data obtained in the example under consideration in terms of bearing capacity instead of settlement leads to an interesting result. Suppose that it was decided to set 0.10 in. as the limiting settlement, and it was desired to find the bearing capacity of a mat foundation, 218 by 252 ft., which would not exceed this limiting settlement. It is assumed that the tests were made on a 1 sq. ft. round plate, with $\frac{P}{A}$ equal to 3.55. The perimeter-area ratio of the mat foundation is,

$$\frac{(218 \times 2) + (252 \times 2)}{218 \times 252} = 0.017$$

* *Proceedings*, Am. Soc. C. E., November 1927, Papers and Discussions, p. 2268.

Let p be the bearing capacity of the large foundation. Using the direct ratio of bearing capacities to perimeter-area ratios:

$$\frac{p}{4\,800} = \frac{0.017}{3.55}$$

and,

$$p = \frac{0.017 \times 4\,800}{3.55} = 23.0 \text{ lb. per sq. ft.}$$

In other words, according to the principle, the mat foundation would not support its own weight at a settlement of 0.10 in.

Another method of analyzing the problem presents itself. Suppose that it is considered desirable to increase the width of the mat foundation by 1 ft., the dimensions would then be 219 by 252 ft. The area has been increased by 252 sq. ft. and the perimeter has been increased by 2 ft. In such a case it would be difficult to convince any engineer that the only additional carrying capacity gained by adding 252 sq. ft. to the area was the shearing strength on an additional 2 ft. of perimeter. Yet that is exactly what the principles under discussion mean, inasmuch as x is the only factor involved in their application.

In spread foundations of practical sizes, the perimeter-area ratio is always small and in such cases to neglect the shearing strength on the perimeter would not lead to serious error. The other factor of strength is the compressive strength of the soil, or the resistance of the pressure bulb to deformation. The total carrying capacity of a foundation due to this second factor of strength is directly proportional to the area. This would seem to indicate that the much criticized practice of using the same bearing capacity for all sizes of footings was, after all, fundamentally sound. This statement could be defended as having considerable merit, inasmuch as it involves less departure from sound principles than the proposition that bearing capacity varies inversely as the lateral dimension.

The greatest criticism that could be made of existing practice in foundations is that the selected allowable bearing capacity is in most cases based on a guess, governed by experiences in previous foundations, and not on the most reliable information. It seems fair to make the statement that, at present, the only reliable method of determining bearing capacity is by actual test.

Inasmuch as it is not feasible to test areas as large as the practical footing, the tests on smaller areas seem to be the only solution. The small areas involve a large perimeter-area ratio and the shearing strength on the perimeter is a very considerable factor of strength. It thus becomes necessary in reducing the bearing capacity to practical sizes, to consider this factor, but not to the exclusion of the other and more important factor of strength due to the resistance of the soil to compression.

The equation for bearing capacity that the writer has found most useful in interpreting test data, and which fits the test data most satisfactorily, includes both factors of strength.

Let p = bearing capacity, in pounds per square foot.
m = shear on the perimeter, in pounds per linear foot.
n = resistance of the soil to compression, in pounds per square foot.
P = perimeter of bearing area, in feet.
A = area of footing, in square feet.
W = total allowable load on the footing.

Then,

$$W = A\,p = P\,m + A\,n$$

and,

$$p = \frac{P\,m}{A} + n \qquad (10)$$

Let the perimeter-area ratio, $\frac{P}{A} = x$; then,

$$p = m\,x + n \qquad (11)$$

A study of the form of Equation (11) reveals that it is a linear function involving two variables, namely, bearing capacity and perimeter-area ratio. If the bearing capacity for a certain type of soil is determined by test for two bearing areas having different perimeter-area ratios, it is possible to solve the two resulting equations for the constants, m and n. Having thus determined the shearing strength on the perimeter and the compressive strength of the soil for which the tests were made, the bearing-capacity equation (Equation (11)) can be used to determine the allowable unit pressure for larger footings such that the limiting settlement will not be exceeded.

In Fig. 23 bearing capacity has been plotted against perimeter-area ratio and the corresponding equations for these curves are noted in Table 6. Curves 1, 2, 3, and 4 are bearing capacities for a 1-in. deflection, which was taken as the allowable settlement. Curves 5 to 10 are for the bearing-capacity limit, which is defined as that unit pressure beyond which progressive settlement takes place. It is the unit pressure beyond which the material acts as a viscous fluid, and the consolidation of the material is not sufficient to permit a condition of equilibrium to be attained with a comparatively small amount of settlement.

These equations and curves have been determined by series of tests on the three different types of soil indicated as yellow clay, stiff yellow clay, and blue clay. This classification is merely used as identification and is not intended to have any other connection with the physical characteristics of the material. These characteristics are given by Figs. 21 and 22.

There are a few points worthy of emphasis in connection with this set of bearing-capacity equations and curves (Fig. 23):

(1) As the size of the footing is increased and x approaches zero the bearing capacity approaches, not zero, but a constant value which is the compressive strength of the soil, and is indicated as the intercept on the load axis.

(2) For a type of soil which has a high degree of fluidity and low cohesive strength the bearing capacity is more nearly constant for all sizes, indicating that x is the least important factor in the bearing capacity.

(3) In practical sizes of footings, having perimeter-area ratios between 0 and 1, the shape of the footing, whether round or square, is not so important. This is consistent with the idea that the loss in bearing capacity (as controlled by the pressure bulb), due to the sharp corners, is a smaller part of the total bearing capacity in large footings. In the smaller areas used in the tests, the loss due to sharp corners is large in proportion to the total area of the plates.

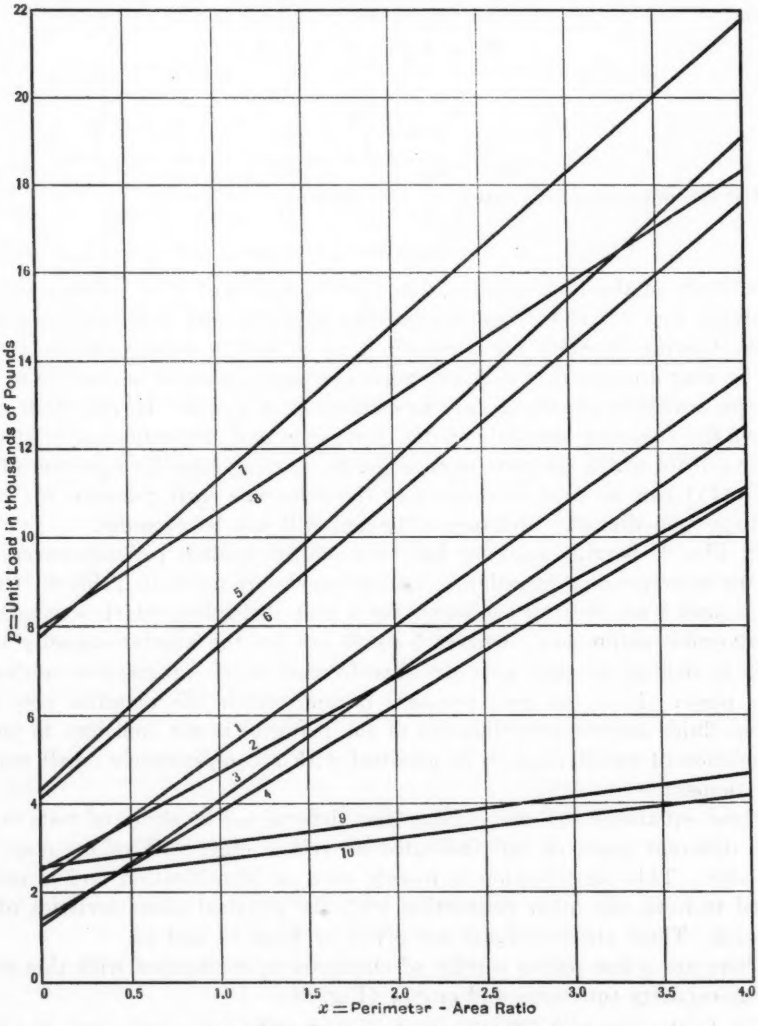

FIG. 23.—BEARING CAPACITY CURVES.

(4) In a type of soil, such as the blue clay, having a high degree of fluidity, the pressure bulb is a controlling factor throughout, and the difference in bearing capacity, due to the shape of the plate, is practically constant. This is clearly shown by the fact that Curves 9 and 10 (Fig. 23) are almost parallel.

TABLE 6.—BEARING CAPACITY EQUATIONS.

Curve No.	Material.	Shape of plate.	Equation.
	BASED ON 1-INCH DEFLECTION.		
1	Yellow clay................	Round.	$2\,880\,x + 2\,260 = p$
2	Yellow clay................	Square.	$2\,150\,x + 2\,530 = p$
3	Stiff yellow clay...........	Round.	$2\,820\,x + 1\,260 = p$
4	Stiff yellow clay...........	Square.	$2\,420\,x + 1\,420 = p$
	BASED ON BEARING CAPACITY LIMIT.		
5	Yellow clay................	Round.	$3\,700\,x + 4\,300 = p$
6	Yellow clay................	Square.	$3\,370\,x + 4\,170 = p$
7	Stiff yellow clay	Round.	$3\,500\,x + 7\,780 = p$
8	Stiff yellow clay	Square.	$2\,600\,x + 7\,950 = p$
9	Blue clay...................	Round.	$520\,x + 2\,580 = p$
10	Blue clay...................	Square.	$490\,x + 2\,250 = p$

In conclusion, it may be stated that the results used in this discussion represent only one comprehensive investigation. It is felt that many such investigations must be carried out before the theories presented may be referred to as principles or laws of soil action. It could hardly be said that anybody understands thoroughly the principles of soil action. "The bulk of the work, the systematic accumulation of empirical data, remains to be done"; and, after that, some of the theories of soil action may become principles and others will be discarded.

FRANK S. BAILEY,* ASSOC. M. AM. SOC. C. E. (by letter).†—It is a very disagreeable experience for an engineer to find after a structure has been erected on a site which he has inspected and given tacit approval, that serious settlement has developed.

The writer remembers one well-educated, intelligent, and able engineer who attained considerable distinction in his special field, who had such an experience with a settling tank designed for the treatment of industrial wastes. A part of the tank settled and cracked and the representative of the company for which it was constructed, made such caustic criticism of the engineer that he, in turn, was filled with resentment and anger. This engineer died at a comparatively early age and a promising career was cut short.

Another well and favorably known engineer who reached still greater distinction, had a similar experience on a larger scale when a considerable portion of the sewage treatment plant which he had designed also settled enough to require much expenditure for repairs. This engineer died before his prime, and in both these cases the disappointment and chagrin caused by the imperfection of their finished work may have been contributory causes to their unfortunately early deaths.

Both these engineers may have given and probably did give what they believed to be a sufficiently thorough study to the foundation problems involved

* Asst. Engr., Metcalf & Eddy, Boston, Mass.
† Received by the Secretary, February 15, 1928.

in the cases cited, but something went wrong, and while it is generally unprofitable to attempt conjecture on what might have been, it is nevertheless natural for one who knew both these men to think that if studies similar to those recorded in the author's paper had been available soon enough, perhaps still greater precautions would have been taken and no troublesome settlement would have occurred.

The writer is impressed by the statement* of such a proficient mathematician as the author is known to be, that,

"Foundation problems, throughout, are of such character that a strictly theoretical mathematical treatment will always be impossible. The only way to handle them efficiently consists in finding out, first, what has happened on preceding jobs of a similar character; next, the kind of soil on which the operations were performed; and, finally, why the operations have led to certain results."

The writer knows engineers who have been responsible for the design of many structures which have been erected and have stood without appreciable settlement. Such favorable results may be in part due to good fortune, but another reason seems to be that it is the habit of these engineers to make a most careful study of the characteristics of the earth on which a given structure is to be built.

Although it cannot be expected that all foundations will be so perfectly designed in the future that no settlements of consequence will occur, it is certain that Professor Terzaghi has acquired much new information that can be applied to improvement in foundation practice, and he is to be thanked for putting this information on record for the use of others as well as for himself.

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2294.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

FLOOD CONTROL WITH SPECIAL REFERENCE TO THE MISSISSIPPI RIVER

A SYMPOSIUM

Discussion*

By Messrs. W. G. Price, Oren Reed, Harold C. Fiske, Charles F. Brooks, William Gerig, C. H. Eiffert, and Sherman M. Woodward.

W. G. Price,† M. Am. Soc. C. E. (by letter).‡—The Mississippi is a river burdened with clay and sand that has flowed in from its tributaries during the ages, to build land in an arm of the Gulf of Mexico.

Until Man began building levees much of the clay and sand was annually spread over the land which the river had built, and which was thus being continually raised. The river developed a channel sufficient to carry only its normal flow of water. Then Man built levees to stop the flood water from spreading over and further building up the land; and these levees, in order to be efficient, had to be high enough to carry all the water of the greatest floods, which is impossible. The engineers in charge reasoned that when the river was confined between levees the increased power of the resulting swifter current would erode and enlarge the channel and increase the carrying capacity so that the height of the levees need not be extended. This reasoning has been proved false.

In 1879 and 1880, during the low-water season, the writer measured, for the Mississippi River Commission, the sand waves in the river at Plum Point and Fulton, Tenn. Many of these waves, which closely followed each other at Plum Point (where the river was very wide and shallow) and at Fulton (where the river was narrower and deep) were as high as 10 ft., with a long

* Discussion of the Symposium on Flood Control with Special Reference to the Mississippi River, continued from March, 1928, *Proceedings*.

† Cons. Engr., Yakima, Wash.

‡ Received by the Secretary, January 12, 1928.

slope on the up-stream side, and they moved toward the Gulf many feet each day.

A large quantity of sand was repeatedly caught in a trap bucket more than 5 ft. above the bottom of the river. The writer also measured the large quantity of silt and sand caught at the surface in the almost continuous boils (which are caused by the river running over the sand waves) in the Lower Ohio and Mississippi Rivers.

At Fulton, where the river was very deep, the many streaks of boiling water, spaced apart (which indicated the location of the crests of the sand waves and their stream lengths), could be seen extending nearly across the river. These data indicated that the scouring and material-carrying power of the Mississippi is very great at low water and immensely greater during floods when the river is confined between levees.

Cross-sections of the Mississippi were made at many places in 1879 and 1880 and these, when compared with sections made since, indicate that the river bed is neither rising nor falling. These data also indicate that the quantity of heavy material being carried in by the tributaries and the quantity supplied by the caving banks is equal to that being dredged and carried out to the Gulf.

The river, by wandering about and digging away its banks as well as some of its levees, is now carrying to the Gulf an immense amount of material which it deposited and stored during past ages. The data also indicate that if the river were stopped from wandering about, and thus scouring material from its banks, the present rate of erosion along the bottom would soon lower its channel and the heights of its floods.

There are only four ways that can be used to control floods in the Mississippi; these are, by the construction of levees, by-passes, spillways (also called outlets), and bank protection to cause the river to dig its bed deeper. Along the river from Cairo, Ill., to the mouth of the Arkansas, levees have failed and they will not give safe protection if built higher. The land has a downward slope away from the river and the flood currents are continually eroding the bank and following down this slope and, therefore, the new levees must be higher.

In this reach neither by-passes nor spillways should be located, therefore, the engineer is limited to the use of bank-protection works. Since 1883 much bank revetment has been constructed by the engineers in charge of this work and it has been destroyed wherever the swift current ran against it.

Practically all these bank-protection works have been of one type, which indicates that the engineers have been unable during forty-four years to produce new or better plans, but have continued to spend appropriations for the construction of worthless designs. The writer's experience in building a different type of durable revetment in the Mississippi indicates that works can be built to protect the present banks and also those against which the swift current from a cut-off would run.

There are several loops in the river where cut-offs can be made, thus creating an increase in the velocity and power to dig the bed deeper.

From the mouth of the Arkansas to the mouth of the Red River, a by-pass through the Tensas would carry enough of a great flood so that the levees along this reach of the Mississippi need not be excessively high and bank-protection works would not be needed, except to protect property and to avoid the continual expense of building new levees. This by-pass should take only the quantity of water required to keep the flood height safely below the top of the levees in the Mississippi; otherwise, it would defeat its object by causing the cross-section of the river to become smaller. It will really be an outlet since its waters will not return to the Mississippi. The by-pass will not close itself. It will form the size of its channel to correspond with the amount of flood water it has to carry, which is the law of all streams in material that is easily eroded.

If the Mississippi were permitted to choose its own route from the mouth of the Red River to the Gulf, it would go by way of the Atchafalaya; but for commercial reasons and to avoid the flooding of valuable plantations along the latter river, this outlet must continue to take only the water that its channel can safely carry with its levees remaining at the present height.

No spillway should be permitted to take water from the Mississippi between the Red River and the Gulf when the water is below the safe flood height. If the spillway is permitted to flow when the river is lower, the section of the channel and the discharge capacity of the river below the spillway will be reduced by a fill and the spillway will not reduce future floods.

Sufficient water should be permitted to flow to the Gulf by way of the Atchafalaya or Vermilion Bay spillways, or by both, as proposed by Mr. J. P. Kemper,* to render spillways into Lake Ponchartrain unnecessary. A wide shallow spillway should be provided below New Orleans, La. This spillway should not lower the flood in the Mississippi enough to cause a fill in the river below the spillway; otherwise, the flood-lowering effect of the spillway at New Orleans would be eliminated. A river in an alluvial land automatically digs or fills its channel to a size proportional to the quantity of water ordinarily flowing through it and it will dig somewhat deeper if its outside curved banks are prevented from caving.

The writer is not familiar with the land through which Mr. Kemper's proposed Vermilion Bay outlet would flow. If it is to be opened, work on it should begin without delay to avoid another disastrous flood in Lower Louisiana. The Tensas and Poydras spillways can be opened quickly when another great flood is approaching.

In prehistoric times, the Red River was not connected with the Mississippi. It flowed to the Gulf by way of what is now the Atchafalaya. Great dams, or what were called rafts by steamboat men, were formed in the Red River and one in the Atchafalaya, above Melville, La. About 1875, the State of Louisiana removed these rafts, but did not make a good job of it, because later two steam tug propellers were broken while trying to pass through the Atchafalaya raft. These rafts were formed of great trees which had caved into the river. Their roots were buried in the bottom of the river and their upper ends were spears,

* In a recent report to the National Flood Commission, entitled "Plan for Control in the Mississippi Valley," by J. P. Kemper, August, 1927, New Orleans, La.

pointing down stream and thus menacing the approach of steamboats. Forty years ago the remains of these rafts in sharp bends in the Red River, the writer noticed, were efficient bank revetments. The Red River, in prehistoric times, having clogged itself by a raft in the Atchafalaya, cut through into the Mississippi and, later, the Mississippi made a cut-off and thus formed Turnbull Island.

The combined outlet of the Atchafalaya and Vermilion must carry the excess of bank-full stage that can safely be permitted in the Mississippi. The Vermilion outlet should have a wide, shallow opening at its upper end so that it cannot take water from the Mississippi below that level, except that it should have a narrow canal deep enough for navigation. This outlet has a slope much greater than that of the Mississippi and, if not controlled, will become the great river, while the Mississippi with its lesser slope, will become a bayou, with a sand-bar filling its channel near Red River Landing.

OREN REED,* Assoc. M. Am. Soc. C. E. (by letter).†—The Symposium on Mississippi River flood control is the most thorough discussion of that great problem that has been published in recent years. The papers were prepared by engineers who have given many years of labor and thought to the control of a mighty river. An exceptional flood has given a full test to the measures of control that have been attempted.

The people of a vast fertile area are asking for relief and protection from flood damage. The country as a whole has at last realized the magnitude of the problems and is in the mood to demand the application of effective remedies for the control of floods in the Mississippi area. It is, in a measure, unfortunate that the engineering features of the problem could not have been more fully decided before Congressional action was started. It is generally admitted that the Mississippi River Commission has made good use of the funds available. Experience has shown that the use of levees has been well justified. However, for physical and economic reasons, levees probably should not be trusted to carry the maximum possible flood which might occur below Cairo, Ill. Reservoirs, spillways, and by-passes should all be studied and used where feasible.

Shortly after the Mississippi flood of 1927, a part of the Duchy of Liechtenstein was covered by the flood waters of the Upper Rhine River. Certain problems developed which were similar to those presented by the Mississippi flood. This Duchy is in the northeast foot-hills of the Alps, and the river at this point has a moderate slope and is bordered by a flood-plain of considerable width.

Levee construction was started about 100 years ago. Since the channel has been fixed between two parallel levees, the stream bed has risen because of the flat gradient and heavy load of débris carried by the stream. In places, the present stream bed is 23 to 29.5 ft. above the land on both sides. At Buchs, where the first crevasse occurred, the stream bed has been raised 11.4 ft. since 1848. Corrective work had been done in recent years in an endeavor to stabilize the channel at its present elevation.

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† Received by the Secretary, January 23, 1928.

Below Liechtenstein, the Rhine flows into Lake Constance. Above the lake there are two levees on both sides. The low-water channel is confined by a low levee, and the flood levee is set back far enough to give ample flood-way. This system of levees is favored because the narrow low-water channel prevents the dropping of the silt and débris load.

The growing danger at Buchs due to the raising of the channel bed was fully understood. The levee on the Swiss side had been raised 1 m. (3.28 ft.) in 1926, and work had been started on the Liechtenstein shore. Two bridges crossed the Rhine at Buchs, a wooden highway bridge and a steel trussed railway bridge. These bridges had not been raised at the time of the flood.

The head-waters of the river were swollen by a severe two-day storm in the last days of September, 1927. At some places, precipitation equalled nearly 7 in. in 2 days. This is, however, not the greatest precipitation on record in Eastern Switzerland. The 2-day precipitation in 1868 at Barhardin Pass was 18.4 in. In 1910, the rainfall was also more intense than in 1927 and covered a greater area.

The quantity of water carried by the Rhine above Lake Constance on September 25 and 26, 1927, was the greatest yet observed. At Buchs, the flood became dammed by the bridges, broke down the weaker highway bridge, and overflowed the Liechtenstein levee above the railroad bridge. When the crevasse occurred, the Swiss levee still had a free-board of 1.3 ft. The break widened quickly, so that a large part of the flood water flowed into the adjacent lowlands. After the high stage was passed, the full flow of the Rhine ran unchecked into the flood-plain, leaving the old river bed dry. The flood waters ran back into the Rhine near the mouth of the Ill River about 10 miles below the break at Buchs. The land side of the levee was washed by the flood waters, thereby causing three secondary breaks.

The influence of the flood on the level of Lake Constance produced no serious results. If a part of the flood had not been held back in the Rhine flood-plain, the level of Lake Constance would have raised about 10 ft. The maximum discharge from Lake Constance was about 141 000 cu. ft. per sec., which was slightly less than in 1910.

It is planned to organize an international commission for the control of the Rhine from the source to Lake Constance and transfer all fees and facilities for river control to this body. A comprehensive plan can then be formulated, which will not be dependent on political boundaries. It is proposed to regulate the mountain torrents so that a large part of the débris will be retained by check dams.

HAROLD C. FISKE,* M. AM. SOC. C. E. (by letter).†—Reservoirs have been created to store water for power purposes and they have been proposed and built for the storage of surplus water at times of flood. It now seems to be taken for granted by many that these two uses are necessarily antagonistic and that if the reservoir is operated successfully for one purpose it will be a failure for the other. Furthermore, it is frequently concluded that the flood

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† Received by the Secretary, January 28, 1928.

reservoir must be totally emptied immediately after local danger has passed, by discharging at the maximum rate which the channel below will carry. The large volume of water thus sent down stream will meet a flood wave in some section below and will increase instead of diminish the flood risks at and below that point.

Mr. Scheidenhelm* is doing a very valuable service in bringing up for further and more careful consideration the assumptions on which these conclusions are based. Very properly, he points out that Nature has given a perfect example of exactly the opposite argument in the Great Lakes where the requirements of both flood prevention and power development are met to an ideal degree. Of course, this well-known example is commonly dismissed at once by comparing the size of the Great Lakes with that of any artificial reservoir that has been or could be constructed by Man, with the value of the territory that would have to be submerged to duplicate the lakes, etc.

What Nature seems to be suggesting is the value of studying her methods and of determining how far Man can go in any given case, not necessarily in duplicating her dimensions, but in securing her results. Since Man cannot hope to match her in all respects he seems sometimes to forget that flood damage may, in general, be prevented by the storage of a part of the excess flow. He forgets, also, that there is some chance to release this storage, not blindly the moment local conditions permit, but after a longer interval. It is clearly impossible to state a fixed rule that will solve this problem to the satisfaction of both flood and power interests under all conceivable conditions, but if each particular case is studied carefully on its individual merits it would seem reasonable to hope that in a certain percentage of instances a solution would be found that was satisfactory to all concerned.

As an example may be cited certain features of the survey of the Tennessee River and its tributaries, authorized by Congress in 1922 and on which a final report is expected early in 1928. This survey is to cover the water resources of the Tennessee Basin and to include full consideration of the needs of navigation, power, flood control, and other allied topics.

One of the many different sections to be studied was the basin of a large tributary, the Clinch River. On this, prior to the survey, private power interests had in mind the construction of a dam 175 ft. high which would create the largest feasible reservoir for power purposes. The correct installation was found to be about 160 000 h.p. In studying this section the survey, in addition to power, gave careful consideration to the needs of flood control, navigation, etc. This is not the place to give a full analysis of all the factors included and all the lines of reasoning followed, but the conclusions are now pertinent and interesting. It may be stated that due attention was given to all items of cost and to all sources of profit.

It was found that below this storage dam six other run-of-river plants were indicated in the next 250 miles, three on the Clinch River and three on the Tennessee River above the existing Hales Bar Dam. By diminishing or shutting off the flow from the Clinch storage during flood seasons the tendency to drown out the three Tennessee plants was reduced and their

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2610.

efficiency at such times was increased. By including in one unit the Clinch storage, the Clinch run-of-river plants, the three run-of-river plants above Hales Bar, the Hales Bar plant, and the plants at Muscle Shoals, there was obtained a development of tremendous size. Its value was greatly augmented by the volume which could be released at low water from the Clinch storage. At flood seasons discharge from the Clinch storage could be diminished or stopped, leaving only the Tennessee River plants in operation. Since the Tennessee would then be at a high stage there would still be ample flow for these plants to operate at full capacity, and the result was substantially the same as that obtained at low water with the entire system operating under the increased flow from the Clinch storage.

As for the flood situation, there were available the daily flow records for a period of twenty years together with incomplete records going still farther back. No attempt was made to discuss or solve hypothetical conditions, but for each day of this 20-year period the output of power and the daily height of the water in the reservoir were determined. This period included one disastrous flood and several that did material damage at Chattanooga, Tenn., about 250 miles below the storage. For one short period, water flowed over the crest of the dam but, as far as floods are concerned, the quantity was negligible. For the remainder of the time the discharge was controlled, and at all times the Clinch River was eliminated as a material factor in Tennessee floods. The height of the storage dam was increased from 175 to 225 ft.

To show the change in technical and general public opinion in this connection a few quotations from official documents may be convincing. House of Representatives Document No. 319* sets forth conditions as they were known before the survey was authorized. House of Representatives Document No. 463† contains a partial report on the survey and outlines some of the information obtained to that time. Referring to the Clinch storage, Document No. 319 gives the following (in terms of installed capacity):

Dam No. 3 (storage).....................	160 000 h.p.
Dam No. 21............................	80 000 "
Total	240 000 h.p.

In reference to the same section, Document No. 463 gives the following (also, in terms of installed capacity):

Cove Creek (storage)..................	200 000 h.p.
Clinton	15 000 "
Melton Hill	60 000 "
Kingston	50 000 "
Total	325 000 h.p.

From this it will be noted that, on this section of the Clinch River, combined navigation, power, and flood studies increased the apparent power possibilities by more than $33\frac{1}{3}\%$ in addition to other flood and navigation benefits.

* 67th Cong., 2d Session, pub. in May, 1922.

† 69th Cong., 1st Session, pub. in June, 1926.

Reporting on the Tennessee River above Chattanooga and its possibilities for run-of-river plants, Document No. 319 gives the following (in terms of installed capacity):

Dam No. 29	36 000 h.p.
Dam No. 30	58 000 "
Total	94 000 h.p.

In addition, it states, "in the entire stretch of the river between Knoxville and Chattanooga, 188 miles long, there appear to be no other favorable power sites."

Concerning the same section, Document No. 463 gives the following:

Coulter Shoals	75 000 h.p.
Marble Bluff	70 000 "
White Creek	150 000 "
Soddy	150 000 "
Sherman	45 000 "
Total	490 000 h.p.

From this it will be noted that on this section of the Tennessee combined navigation, power, and flood studies increased the apparent power possibilities by more than 400%, in addition to the other important flood and navigation benefits. It developed, for power, the entire head between Knoxville, Tenn., and Chattanooga.

Prior to the survey, power companies were interested only in Dams No. 3 and No. 21 (Document No. 319 list) on the Clinch River, and this interest was largely academic. When Document No. 463 was published, power companies had already filed with the Federal Power Commission applications for preliminary permits for the nine sites listed for both the Clinch and Tennessee Rivers, as well as for others.

In conclusion, it is submitted that the effect of combined power and flood use of storage should never be prejudged, but each case should be investigated carefully and judged strictly on its merits. Mr. Scheidenhelm is doing a real service to the public in bringing this phase of the matter before the Society for discussion.

Charles F. Brooks,* Esq. (by letter).†—Dr. Frankenfield presents the average rainfall by water-sheds in the Mississippi Basin and details the rainfalls by months prior to some of the great floods. Comparing 1927 with 1922, he indicates‡ that the first four months of both years had about the same excess of rainfall, but that the greater flood in 1927 was due to the greater excess in April. Such being the case, it seemed worth while to study these April rains in relation to the weather map situations both local and general.

* Prof. of Meterology and Climatology, Clark Univ., Worcester, Mass.

† Received by the Secretary, February 7, 1928.

‡ *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2483.

The results of a small investigation on these lines jointly by Nesbitt H. Bangs, Graduate Student, Clark University, and the writer are summarized herewith.

Although the Mississippi flood, like the recent New England flood, was caused by the importation of water vapor from tropical waters precipitated in excessive amounts over a region that is extended in area, it was not the result of a single importation. Rather was it brought about by a series of such importations extending back to the beginning of August, 1926, but culminating in the astonishing amounts of rain which fell in the Middle Mississippi Valley in April, 1927.

For a large area comprising Southern Illinois, Eastern Kansas, Eastern Oklahoma, Western Tennessee, Missouri, Arkansas, and Louisiana, the rainfall averaged more than twice the normal. The precipitation on parts of this region was far more than this, as for example, Arkansas with a rainfall of 12.93 in. compared with a normal of 4.88 in., and Western Tennessee with a rainfall of 11.35 in. compared with a normal of 4.40 in. Curiously enough, the State of Mississippi as a whole experienced only the normal rainfall, but certain stations in the northwestern counties recorded rainfall far in excess of the average, notably Greenville, with a rainfall of 13.67 in., an excess of 9.06 in. above the normal. Table 37 shows the rainfall in the various States and the departure from normal.

TABLE 37.—Rainfall, in Inches, April, 1927.*

	April, 1927.	Normal.	Departure.
Arkansas	12.93	4.88	+8.05
Western Tennessee	11.36	4.40	+6.96
Eastern Oklahoma	9.48	5.47	+4.01
Missouri	8.46	3.73	+4.73
Southern Illinois	8.43	3.92	+4.51
Eastern Kansas	7.40	3.05	+4.35
Louisiana	6.85	4.70	+2.15
Mississippi	5.12	5.19	±0.07

* From *Climatological Data,* April, 1927.

The rainfall for the month was not only intense, but was also remarkably persistent. From April 1 to April 22, rain fell in considerable amounts every day in some part of the region. Only on April 2 and 3 did Arkansas, for example, report no precipitation during this period, but on those days rain was falling in Illinois. Table 38 shows the periods when the rainfall was general and copious in the different sections of the region.

Table 38 shows five well-defined periods of rainfall: the 1st, 5th to 9th, 7th, 11th to 16th, 18th to 21st, and 29th. A study of the weather maps for these periods indicates a close analogy between the conditions causing the rain on the 1st and 29th, the rainfall on the 5th and 18th to 21st, and indicates that the rainfall during the long period from the 7th to 16th was caused by a pressure distribution that remained persistently the same throughout the period.

Any conditions that will bring about the rapid ascent and consequent rapid cooling of warm moist air will cause heavy rain, and these conditions

were particularly suitable during most of April. Pressure conditions over the United States and the Atlantic Ocean, as shown by maps,* were such that the southerly and southeasterly winds blowing up the Mississippi Valley originated, not merely over the Gulf of Mexico, but far to the southeastward over the Caribbean; hence they were warmer at the start, and passing as they did over such a great expanse of the warm Equatorial Current were given an amount of water vapor that was greater than normal through a considerable depth of winds. Arriving in the Mississippi Valley, these moisture-laden winds were forced to rise first over the mountains of Arkansas, and, more important, over great wedges of cold dense air pressing in from the north-east, north, and the northwest. No wonder the rainfall was heavy.

TABLE 38.—Rainfall Periods.

Days of month.	April.																													
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
Arkansas	*	.	.	.	*	*	*	*	*	.	*	*	*	*	*	.	.	*	*	*	*	.	.	.	.	.	.	.	.	.
	*	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.
Western Tennessee	*	.	.	.	*	.	.	*	*	.	*	*	*	*	*	*	.	.	.	*	*	*	.	.	.	.	.	.	.	*
Eastern Oklahoma	.	.	.	.	.	.	*	*	.	.	*	*	*	*	*	*	.	*	*	*	*	.	.	.	.	.	.	.	.	.
Missouri	*	.	.	.	.	.	*	*	*	.	*	*	*	*	*	*	.	*	*	*	*	.	.	.	.	.	.	.	*	.
Southern Illinois	*	.	.	.	*	.	*	*	*	.	*	*	*	*	*	*	.	*	*	*	*	.	.	.	.	.	.	.	*	*
Eastern Kansas	*	.	.	.	.	.	*	*	*	.	.	*	*	*	*	*	.	*	*	*	.	.	.	.	.	.	.	.	.	.
Louisiana	.	.	.	.	*	*		(north)		.	.	.	.	*	*	*	.	.	.	.	*	.	.	.	.	.	.	.	.	.

Given then this general picture of a vast flow of air from tropical waters meeting cold dense air from the higher latitudes, it is interesting to note under what local conditions heavy rainfall ensued. In general, the rainfall was of three types: (*a*) That which occurred in warm, southerly surface winds on the southeastern quadrant of a low-pressure area; (*b*) the warm-front type; and (*c*) the cold-front type.

(*a*).—In this case the lower wind coming directly from the tropics becomes warmer much more rapidly than the upper wind, which is usually from some westerly direction. Instability and consequent violent overturning of the surface air results, forming thunderstorms. At times, when the wind aloft becomes much colder from its source in a cold southwestern quadrant of the low-pressure area, the unstable overturning of the warm southerly wind below may reach cloudburst and tornadic violence.† The great number of heavy thunderstorms and tornadoes over the region bears witness to the frequency of this occurrence. Of forty tornadoes reported during the month nearly all occurred over the region discussed.

(*b*).—The second type of rainfall which fell when the warm southerly wind was forced to rise over a cold northeasterly surface wind flowing out from an area of high pressure to east and northeast, the so-called "warm-front" type, is usually a combination of general ascent and instability rainfall.

* In *Monthly Weather Review*, April, 1927.

† "Elementary Meteorology," by W. M. Davis, Bost., 1894, p. 258.

(*c*).—The third type occurred when the front of an energetic high pressure area pushed southeastward with cold northwest winds under-cutting or over-cutting it and forcing the warm, moist surface air upward. This is called the "cold-front" type and the rain falls with northwesterly winds at the surface and with rapidly falling temperatures. Where the over-cutting runs 50 or more miles in advance of the cold front at the surface, this type merges with the violent aspects of the southerly type.

What were the weather map situations of April that brought about the rainfall periods previously noted? On the morning of the 1st a low-pressure area of considerable intensity was centered over Missouri and Illinois and heavy rain fell with southerly winds over Missouri, Illinois, and Western Tennessee, averaging 1 to 2 in. in Southern Illinois. Here was an example of the first type; that is, heavy showers in southerly winds caused by overturning of the warm moisture-laden air.

On the 5th, an area of low pressure that had been centered over Oklahoma the preceding day had moved northeasterly to Lake Superior, and a wedge of cold air with moderately high pressure had cut in to the southwest of the low-pressure area. Heavy rain fell with northwest winds in Mississippi, Missouri, Arkansas, and Louisiana (Shreveport, 2.82 in.). This was an example of the cold-front type caused by the crowding in of cold air to displace the warm moist air of the preceding southerly winds.

On the 7th an area of high pressure of great magnitude appeared over the Province of Ontario, Canada, and the Lake region. An ill-defined area of low pressure was centered over Texas, and this type of map persisted until the 16th. A great mass of cold air, constantly reinforced, was moving southeastwardly from Manitoba, Western Ontario, and the Lake region. Throughout this period there was a constant conflict between the cold surface northeast winds flowing from the high-pressure area and the warm southerly winds flowing northward in the Mississippi Valley on the easterly side of the low-pressure area in Oklahoma. It is difficult to indicate the exact point at which the northeasterly winds ceased and southerly winds commenced, but at St. Louis, Mo., and Cairo, Ill., there were apparently seven days of rain with northeasterly winds; at Memphis, Tenn., four; and at Vicksburg, Miss., one, showing that the line of maximum rainfall with northeasterly winds was somewhere between Cairo and Memphis.

North of this line the surface wind was northeast and the rainfall was of the warm-front type, caused by the warm wind ascending above the cold wedge flowing out from the northeast high-pressure area; but south of the line, the rain that fell was of the instability type, (*b*).

An interesting variation of these types occurred during this period, especially on April 13-14, when the Oklahoma low-pressure area shifted slightly to the northward, as a tongue of cold air swept over Texas. Heavy rains of the cold-front type fell with northwest winds over Texas on these days (Dallas, 2.06 in. on the 13th). In the period, April 10 to 14, twenty tornadoes occurred, most of them in Texas. When the cold front, now in the free air, reached New Orleans, La., on the 15th, 14.00 in. of rain was precipitated.

On the 19th an energetic area of high pressure commenced to speed southeastward from the Northern Rocky Mountain region toward the Gulf of Mexico, and for the next three days (19th, 20th, and 21st) during its passage, extremely heavy cold-front rains (5 to 12 in.) fell in Arkansas.

The final period of heavy rain came on the 29th when an area of low pressure very similar to that of the 1st passed eastward over Missouri. The rainfall was particularly heavy in Illinois, Missouri, and Tennessee (on the 30th).

Thus, from the general description just presented, it can be seen that the rainfall periods bore certain resemblances to one another. For instance, those of the 1st and 29th occurred with southerly winds; those of the 5th and 18th to 21st were of the cold-front type, falling with northwesterly winds; and those of the 7th to the 16th were generally of the warm-front type, falling along the line where the warm winds on the eastern side of the low-pressure area began to override the cold air from the high-pressure area to the eastward. An attempt has been made to separate the total precipitation at a group of selected stations into the amounts that fell with northeasterly, southerly, or northwesterly winds.*

To assume that all the rain which fell during the previous 24-hour or 12-hour period fell with the wind from the direction indicated at the end of the period is possibly to stray appreciably from the actual facts, although the persisting character of the weather map situations from day to day in these rainy periods make the results seem probably significant.

Over the flooding rain region as a whole, as represented by nine stations, 43% of the 12-hour rainfall (8:00 P. M.-8:00 A. M.) came in the region of southerly surface winds, 35% in the northeasterly sector, and 22% in the region of northwesterly winds. The corresponding 24-hour values are almost exactly the same. 42, 34, and 23 per cent.

It is interesting to note that the more northerly stations had the greater amount of rains of the warm-front type. Thus, St. Louis and Cairo had 48% and 41%, respectively, of this type, whereas Shreveport and Vicksburg had only 2 and 7 per cent. This was to be expected, considering the pressure distribution already noted; but whatever the type, it is obvious that the excessive rains were due to the large importation of vapor from tropical waters.

Conclusion.—The April contribution to the Mississippi flood was not a single paroxysmal rainstorm, such as produced the New England flood in November, 1927, but a number of heavy rains at frequent intervals over substantially the same region. The vapor for these rains came day after day in a great flow of air from Southern waters. Suffering little depletion on the coast, this air converged in the central portion of the Lower Mississippi Basin and was there also forced to rise not only over the mountains of Arkansas, but also over great barriers of denser air pouring out of the northeast, north, and northwest. Furthermore, as the frequent thunderstorms bear witness, so great were the temperature contrasts in short distances, both verti-

* Acknowledgment is made to Mr. A. J. Henry, of the U. S. Weather Bureau, for the manuscript 12-hour precipitation charts, and to Mr. Thomas R. Brooks, of the Weather Bureau, for kindly copying a number of the 8.00 P. M. weather maps for April, 1927.

cally and horizontally, that the warm humid air was forced violently to great heights. These several causes of ascent and, therefore, cooling and drying of enormous masses of humid air acted over practically the same regions for days at a time, and, in consequence, produced the tremendous rainfalls that were chiefly responsible for the great flood.*

WILLIAM GERIG,† M. AM. SOC. C. E. (by letter).‡—Major Godfrey has presented an excellent paper on the subject of the improvement of navigation and flood control,§ and has correctly described the characteristics of the Lower Mississippi River, particularly during the low-water stages when navigation improvements are necessary.

A channel 9 ft. deep has been maintained during the past thirty years by dredging between Cairo, Ill., and the mouth of the Red River. This channel is temporary and is dredged, on some of the bars or crossings, during each low-water period. The depreciation of the channel is caused primarily by material deposited on the bars from caving banks immediately above the troublesome crossing. If, therefore, the banks of the river are stabilized, the channels in the crossings will unquestionably become more fixed as the quantity of material carried by the river is considerably reduced. The river will then cease, to some extent at least, its "restlessness".

The low-water channel in certain localities, even after the banks are stabilized, will require some dredging or regulation in order to hold it in a definite location. The regulation can probably be done, to some extent, by contraction works of proper design. The writer believes that considerable experimental work should be done in order to develop, at a reasonable cost, an effective type of contraction works for the lower river.

Since 1883 many types of contraction works have been tried on the Mississippi River below Cairo. They consisted of permeable pile and brush dikes, abattis dikes, massive rock-filled brush dikes, sand dams, etc., all of which were either total failures or have proved to be only temporary. These types were successful on the Upper Mississippi and other rivers. However, if the banks are stabilized well in advance of the construction of contraction works, a proper low-water type will no doubt be developed.

It has been computed that the Mississippi River expends at the rate of 60 000 000 h. p. of energy or heat during great floods in its course from Cairo, to the Gulf of Mexico. Some of this energy is, of course, expended in the erosion of the river banks and in the transportation of material in suspension. If bank erosion is curtailed, the energy saved will possibly be utilized to deepen the channel. The writer, however, does not believe that such an improved low-water channel will have any marked effect on flood heights, except that they may be slightly reduced in floods where the condi-

* For a full discussion of the Mississippi flood see *Monthly Weather Review*, Supplement 29, by H. C. Frankenfield and others, summarized in *Monthly Weather Review* for October, 1927, pp. 437–452, 10 diagrams and photographs, and very briefly in *Science*, January 5, 1928, pp. 15–16.

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‡ Received by the Secretary, February 17, 1928.

§ *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2557.

tions are similar to those of 1927. During that year the river was at or above flood stage for many months prior to the great rainfall in the Lower Valley of the Mississippi River during March and April, and at no time did the river fall sufficiently to permit erosion of the bar channels that had been filled. Erosion of sand-bar obstructions in the channel begins soon after the stream has precipitated some of the material in suspension. Precipitation starts when the peak of the flood is reached and continues as the river falls, and as the river is relieved of the work of transportation, it applies this energy to providing a channel across the bar obstructions between the pools. The Mississippi River has one characteristic that is common to almost all streams whether large or small, that is, it is composed of a series of pools separated by shoals and rapids. This condition (high bars across the channel) has decreased the slope of the river to some extent, and at Cairo, and other points, has caused somewhat higher stages for a given discharge, than would have occurred under normal conditions or with stabilized banks.

Again, when the banks are stabilized, the depreciation of the channel below a crevasse will be decreased, for the reason that the river is not transporting such large quantities of material in suspension, its source of supply has been considerably curtailed, and, hence, there will be little difference between the effect on the low-water channel of a controlled spillway or weir and that of the uncontrolled spillway.

The writer agrees with Major Godfrey that some minor straightening of the low-water channel, as long as the general regimen of the river is not affected, may be advantageous in some localities. Careful study, however, should be given such proposals before they are undertaken, or otherwise the high-water slopes may be affected.

C. H. Eiffert,* M. Am. Soc. C. E. (by letter).†—Mr. McCarthy states‡ the weight of dry forest litter at the Wooster, Ohio, Station to be from 1 827 to 17 545 lb. per acre and that it will absorb 1½ to 2½ times its dry weight in water. The weights were obtained in the fall before the leaves had fallen. The annual deposit of litter is estimated to be about 2 tons, dry weight. The average weight of dry litter per acre would be about 5 tons. Add to this the annual crop of dry leaves and there would be about 7 tons of litter per acre to absorb water during the following flood season. Assume the absorptive capacity of this litter to be two times its dry weight, or 14 tons per acre. This would amount to 450 cu. ft. of water, or 0.12 in. of rainfall. The Ohio Basin is estimated to be one-third forested. (The area that might be reforested, although it amounts to considerable in acres, will affect this proportion very little.) If 0.12 in. were absorbed by the forested area, it would amount to 0.04 in. over the entire Ohio Basin. The absorptive capacity of the litter when dry is assumed to be 14 tons per acre. It will never be dry during February, March, or April, when the great flood-producing rains occur. Assuming

* Chf. Engr. and Gen. Mgr., The Miami Conservancy Dist., Dayton, Ohio.

† Received by the Secretary, February 24, 1928.

‡ *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2514.

the quantities of litter stated to be the average, the net effect of the forest litter on run-off may be considerably less than 0.04 in. in the Ohio Basin in a great flood.

In the quotation* from "The Relation of Forests and Water" by Zon, it is stated that forests retard the melting of snow in the mountains. Whether or not this retarding action helps to reduce floods is questionable. The forest in this case acts as an uncontrolled reservoir and may release its storage at the wrong moment.

It is very difficult to take the results of experiments such as those being made at Wagon Wheel Gap, Colo., in the White Mountains, or in Switzerland,† and try to deduce therefrom possible results in the Mississippi drainage basin. These are small areas. Their geology, climatic conditions, altitudes, and geographical locations differ greatly from the parts of the Mississippi Valley contributing most of the flood run-off. The Swiss experiments were referred to in order to show the difference in run-off between forested and deforested areas during a violent summer storm. These experiments seem to be the most thorough and complete that have been made along this line, and the results for a period of thirteen years have been reported by Dr. A. Engler. Some of the conclusions reached are as follows:‡

"The run-off due to heavy rainfalls of short duration as thunderstorms or cloudbursts is much less from the forested than from the deforested area, peak discharges from the forested area being from one-third to one-half of those from the deforested tract and the total run-off for this class of storms from the former is usually about one-half that from the latter.

"The run-off due to long continued rainfall is greatly affected by the water content of the ground prior to such rainfall. In case this water content is considerable the forest will have no effect, the run-off being the same from forested as from deforested areas.

"Over the entire period of 13 years the run-off factors for the two areas are practically identical, being 60.0% for the forested and 60.4% for the deforested area."

The writer wishes to emphasize the matter of the water content of the soil or its absorptive capacity by an illustration from the Miami Valley. This valley may be considered as deforested. During the storm of September 12 and 13, 1925, the average rainfall north of Dayton, Ohio, over an area of 2 525 sq. miles, was 3.20 in. The run-off was 0.04 in., or 1.25% of the rainfall. During the storm of March 19 to 21, 1927, the rainfall was 3.44 in. and the run-off resulting from it was 2.31 in., or 67% of the rainfall. The first-mentioned storm was of shorter duration and other factors being equal, would have given the greater run-off. The maximum stage reached at Dayton during the first-named storm was 2.9 ft. while during the latter it was 12.8 ft.

Unquestionably the difference in run-off between these storms was due to the difference in soil conditions. The storm of September, 1925, came at the end of the growing season and following a dry period during which the transpiration and evaporation had been abnormally great. At the beginning

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2511.

† *Loc. cit.*, p. 2515.

‡ The writer's translation.

of this period there was a deficiency in rainfall of about 5 in. since the first of the year. These factors combined to give the soil a tremendous storage capacity.

Preceding the storm of March, 1927, there had been little evaporation, and there was no growing vegetation to consume moisture. To November 1, 1926, there had been an excess of 5.79 in. in precipitation since the beginning of the year.

It seems to the writer that while the absorptive, retentive, and retarding effects of the forests have been greatly emphasized by those who advocate reforestation as a means of assisting in flood control, the same factors in regard to pastures, growing crops, or well cultivated fields, have been generally neglected or entirely disregarded. The comparisons made are too often between forests and denuded or barren hill and mountain sides. That part of the Mississippi Basin east of the 100th Meridian yields more than 95% of the Mississippi flood water. The part of this area which is not forested, consists principally, not of barren hillsides, but of well tilled farm lands which have a considerable capacity for retaining moisture.

While there may be a small surplus of farm land at the present time, no one would advocate returning good agricultural land to forest conditions. On the other hand, there can be no valid objections to reforesting the denuded hills, not for flood-control purposes, but for the products of the forest. Let forestry stand on its own merits. Even if it could be shown that forests might have an important influence on great Mississippi River floods, the areas which might be available for reforestation would not be great enough to alter materially present conditions.

The forest will undoubtedly prevent, to a large extent, the washing away of the soil from the hillsides, but this is not necessarily an argument in favor of reforestation for flood control. The Missouri experiments* seem to show that it will require 437 years to erode the 7-in. surface layer from land planted with a rotation of corn, wheat, and clover, and 3 547 years to erode 7 in. from land in blue-grass sod. One may see gullies and other signs of erosion even in the forest. Will it take longer than 3 547 years to take 7 in. of soil from forested land?

Referring to Colonel Kelly's paper,† the writer desires to call further attention to the possible effect of the Miami River flood-control works on floods in the Ohio River.

In Fig. 46 are shown hydrographs of the actual 1913 flood in the Ohio, at Cincinnati, the 1913 flood in the Miami River, at Hamilton, a controlled 1913 flood, at Hamilton, and a 1913 flood in the Ohio with the peak in the Miami, occurring on April 1 instead of March 26. The actual figures at the mouth of the Miami River would exceed those shown herein. Information for completing hydrographs for both streams at the mouth of the Miami was not available; however, the curves shown answer the purpose of illustrating the

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2514.

† *Loc. cit.*, p. 2519.

point it is desired to emphasize. Comparing the curves for the actual and the controlled 1913 flood, at Hamilton, it is seen that the maximum excess of the controlled flood over the actual is 40 000 sec-ft. and occurs on March 29. The maximum possible increase in the Ohio, therefore, would be 40 000 sec-ft. for such a flood and would occur if this stage of the Miami flood occurred simultaneously with the maximum stage in the Ohio. Under any other conditions the increase would be less than 40 000 sec-ft. Suppose, however, that the peak in the Miami, instead of coming on March 26, should synchronize with the Ohio peak occurring on April 1. The uncontrolled Miami River flood would then increase the Ohio peak by 325 000 sec-ft., but controlled as at pressent this increased peak would be reduced 200 000 sec-ft. In other words, in a flood like that of 1913 controlled by the retarding basins, the maximum possible increase in the Ohio River would be 40 000 sec-ft., while the maximum possible reduction might be as much as 200 000 sec-ft.

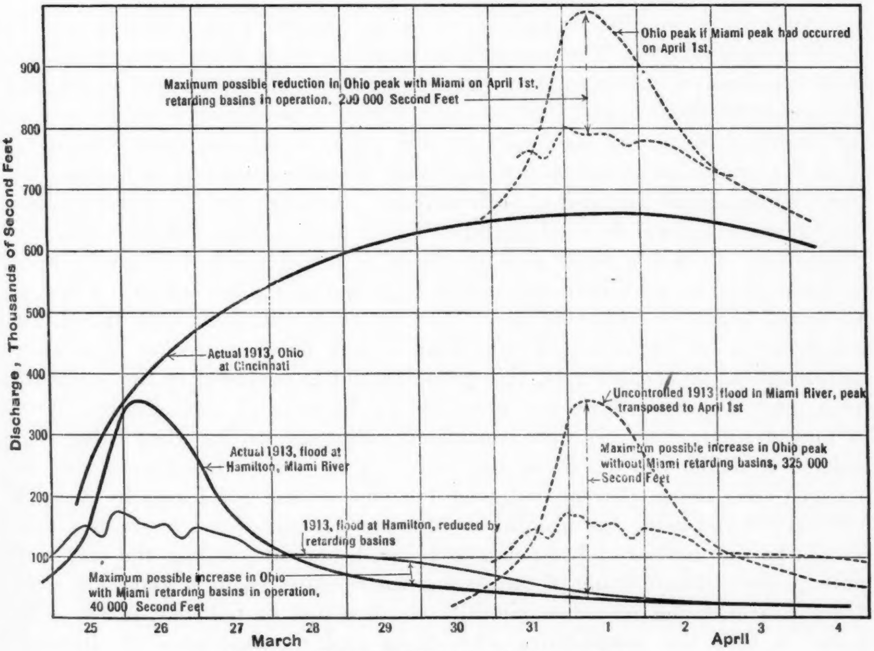

FIG. 46.—HYDROGRAPHS OF THE 1913 FLOOD.

It cannot be safely argued that because there is no record of this combination, it cannot occur. There are no definite previous records of floods as great as that of 1913 on the Miami, or like that of 1927 on the Mississippi. Before they occurred, they were thought by many to be impossible. Yet certain combinations made them possible.

The writer does not wish this to be understood as an argument in favor of reservoirs on the head-waters of Mississippi tributaries. He simply desires to state a possibility which has not heretofore been emphasized.

SHERMAN M. WOODWARD*, M. AM. SOC. C. E. (by letter).†—The paper‡ by Colonel Kelly serves as an admirable illustration of the urgent necessity for the creation of a fact-finding commission, authorized to use ample time and money, to whatever extent may be necessary, in order to investigate and determine the best solution of the Mississippi River flood problem.

If engineers had accurate maps of actual important reservoir sites with data showing storage capacity at various depths; if they had detailed estimates of quantities and costs covering the dam, spillway, control gates, and flowage rights, all accompanied by complete flood hydrographs of the stream bearing the reservoir site and other information needed to show the time relation of the storage to the flood flow in other tributaries, and to the regions to be protected; if they had such actual information, it would be possible to discuss, much more satisfactorily, the methods used for determining the cost of reservoir control, the comparative value and merit of such control, and the reliability of the conclusions reached.

As it is, it may be profitable to consider, briefly some methods used in making such studies. The approximate studies already made seem to indicate that complete protection might be secured by levees alone at a cost of approximately $1 000 000 000, and that an equal degree of protection could be secured by reservoirs at a cost somewhat greater. The wisest and most economical plan for protection would then undoubtedly be some combined plan using the most advantageous features of both methods. To work out the best combined plan requires careful detailed consideration of each feature and each reservoir separately. In such a study, to lump the effect of a number of reservoirs or to average them, would vitiate the results. The last or top foot of such a system of levees might cost $100 000 000. If a group of reservoirs were found, the construction of which would contribute to the flood protection at a more advantageous or cheaper rate, this group should be included in the combined plan, and the expenditure on levees should be reduced accordingly. Even if only a single reservoir were considered advantageous, its effect being considered equivalent to 0.1 ft. on the height of the levees, the levee should be lowered that much, thereby reducing the expenditure for levees by perhaps $10 000 000 and using the needful amount for the cost of the reservoir. This should be the method of balancing to be used, notwithstanding the fact that perhaps the ultimate fixing of the levee grade remains finally a somewhat arbitrary matter, being placed empirically at some foot or half-foot mark. The point is that the comparative advantages and disadvantages of different features can be and should be determined with more precision than may be claimed for the finally adopted dimensions.

The estimates so far published for the construction of floodways are vague and uncertain. When these estimates are satisfactorily settled, the same plan of comparison may be equally well applied to them.

It would be highly desirable to build at least one large reservoir or one group of reservoirs conveniently situated, for flood protection on the Lower

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† Received by the Secretary, March 1, 1928.

‡ *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2519.

Mississippi, if merely as a demonstration or experiment and for the purpose of testing the operation of the reservoir during flood times. This is a field in which there is very little experience as a guide.

What Colonel Kelly* calls dependable operation of flood-control reservoirs, is storage at a uniform rate for a period of 90 days, entirely irrespective of flood conditions on other tributaries, or any other existing condition. This is perhaps the least efficient system of operation that can be imagined and still be called a system of rational control. It seems probable that reservoirs close to the region to be protected can be operated three times as efficiently. Colonel Kelly bases most of his estimates of comparative value on averages of large groups of reservoirs. To determine the most economical combined system, the comparison should begin with the least expensive single reservoir measured in units of flood protection, and then should proceed regularly toward the more expensive. Taking all these items into consideration, it appears probable from Colonel Kelly's figures that a considerable number of reservoirs will be desirable and economical in a combined system for flood protection. It is impossible to determine this point satisfactorily or conclusively until more detailed and accurate data are available.

The other point to be considered is the question of the advantage of close proximity of the reservoir to the area to be protected. It is a great advantage to have some of the reservoir storage near the protected area, but it should be understood that it is not necessary or particularly beneficial to have all the reservoirs close together. This can most easily be made clear by a simple example. Assume two equal reservoirs on the same or different tributaries with adequate flow, Reservoir A being close to the protected area and Reservoir B, two weeks distant. It can be shown that, under rational control, they can in general be operated as efficiently as if both of them were as close to the protected area as Reservoir A. Assume that on a certain date conditions make it desirable for the flow below Reservoir A to be reduced by a constant amount, such that when stored in Reservoirs A and B it would just fill both of them in the eight weeks. Let x = one-eighth of the capacity of Reservoir A. Then, to effect the desired protection below Reservoir A, $2x$ is the quantity of water per week by which the flow below it is to be reduced. The operation of the controlled reservoirs would be as follows.

On the given date, begin storing water in Reservoir A at the rate of $2x$ per week. This would reduce the flow below Reservoir A by the desired amount. On the same date begin storing water in Reservoir B at the rate of $\frac{4}{3}x$ per week. At the end of 2 weeks the reduced flow below Reservoir B would reach that of Reservoir A. At this time change the rate of storage in Reservoir A to $\frac{2}{3}x$ per week. This rate of storage combined with the reduced rate of flow from Reservoir B would maintain the desired reduced flow below Reservoir A. At the end of 6 weeks from the beginning, Reservoir B would be full, and the full stream flow would be allowed to pass, but

* *Proceedings,* Am. Soc. C. E., December, 1927, Papers and Discussions, pp. 2520–2521.

this flow would not reach Reservoir A until 2 weeks later, that is, at the final end of the 8-weeks period. At this final date Reservoir A would also be completely filled, having stored at the rate of $2x$ for the first 2 weeks and at the rate of $\frac{2}{3}x$ for the next 6 weeks, making a total final storage of $8x$ in each reservoir.

This simple example is only to illustrate the general method. Whatever the sizes and locations of the various reservoirs in a system, they can be similarly operated in co-ordination with each other with perfect elasticity to the capacity of the stream supplying each reservoir.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

EMERGENCY DAM ON INNER NAVIGATION CANAL AT NEW ORLEANS, LOUISIANA

Discussion*

By Messrs. R. O. Comer, W. J. Barden, Roger D. Black, and Samuel McC. Young.

R. O. Comer,† Esq. (by letter).‡—The author has presented an interesting and valuable summation of the efforts that have been made to devise machinery to check the uncontrolled flow of water in locks and other large channels. Furthermore, he has given a specific and accurate description of how this has been done in a relatively cheap, simple, and effective way.

Devices of this kind are few in number and the cases are fewer still where these devices have been put to use under emergency conditions. The time is probably quite remote when any considerable amount of data will be derived from lock accidents. The designer in this problem is obliged, therefore, to visualize every possible mechanical weakness or defect and to provide against an extended chain of events in a calamity which he, in all probability, will never see.

In comparing the stop-log type of dam and the wicket-girder type, the writer's remarks are based on observations made on the Panama dams and the New Orleans Dam in each case. The stop-log type of dam is favored over the wicket type, under the conditions ordinarily met with in lock design, for the following reasons:

1.—The bridge, in the general design and in the details, is simpler. The hoisting machinery is likewise more simple. On the other hand, the wicket girder dam is a very complicated machine both structurally and mechanically.

2.—The cost, as shown by Mr. Goldmark,§ is less.

3.—The time necessary for planning, fabrication, and erection, is less.

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* Discussion of the paper by Henry Goldmark, M. Am. Soc. C. E., continued from February, 1928, *Proceedings*.

† Syracuse, N. Y.

‡ Received by the Secretary, January 10, 1928.

§ *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2640.

4.—The maintenance should be less. There are relatively few parts in the stop-log girder type of dam to get out of order. The details are generally simple and rugged and, with few exceptions, all parts are readily accessible for painting, inspection, and repair. However, as far as accessibility is concerned, there is little choice between the two types.

5.—The number of men required for operation is less. The stop-log girder dam is a one-man operated machine in the emergency case. No great amount of skill is required. On the other hand, the wicket-girder dam seems to require a crew of about fourteen or fifteen well-trained men for good results.

6.—The stop-log girders can be adopted for use as a lock caisson with small extra expense.

7.—The weight of the loaded bridge under the same conditions will no doubt be less, since only a part of the dam proper is carried at one time. This was no small advantage at New Orleans because of the low bearing value of the soil and because of the fact that the dam and its foundation necessarily formed an unfavorably located concentrated load on one wall of the lock.

8.—The leakage rate is a great deal less. At the time of the test at New Orleans, full head was artificially created on the up-stream face by pumping the forebay full of water to approximately the 1922 high-water mark. The entire down-stream face was bared by pumping. Under these conditions, the leakage was so small that no one thought it worth while to go to the trouble of measuring it. It was thought at the time that between 100 and 200 gal. per min. would account for all of it. The leakage was confined almost entirely to a number of open spaces in the steel horizontal sealing strips between the upper girders. The strips of the lower girders appeared absolutely tight, very likely because of the superimposed weight of the upper girders. The vertical seals in the lock walls were likewise quite tight, not only because of good workmanship, but because of the large unit pressures on the sealing surfaces. Before the test, when the girders were piled up across the lock in the dry, the joints were inspected for tightness by noting the amount of light coming through and testing the openings with strips of thin flat steel. There were only a very few places that showed more than 0.02-in. opening. It is recalled that perhaps 50 or 60% of the length of the seals showed no light coming through at all. Under the heat of the sun, the upper girders would be bowed up so that they had a bearing only at the ends.

The steel sealing strips were made in short sections to conform with the down-stream curved outlines of the girders. The strips were fitted with adjusting or leveling screws and were backed up with Babbitt metal. Obviously, this construction made for accuracy.

It can be readily appreciated that the workmanship in lining up the sealing strips was unusually good. The records show that the leakage under a similar test at Panama was 950 cu. ft. per sec. This can be regarded as a practicable minimum for the wicket-girder type of dam. No particular effort was made at the time of the New Orleans test to treat the sealing surfaces. There was the usual coat of slush oil on them which had been applied some weeks previously. The girders for the purposes intended can be pronounced tight. It seems that the degree of tightness stated is about the best it is practicable to

secure using sealing strips of steel. If in future work a greater degree of tightness is desired, the design of the seals very likely can be changed. However, some sacrifice in durability would no doubt have to be made.

9.—It is obvious that the most essential requirement of an emergency dam is certainty of action. In the wicket-girder type the multiplicity of relatively small leaves with numerous sliding surfaces offer an opportunity for jamming. The leaves at Panama jammed on some occasions during routine drills, at least in the earlier days of operation. The one instance of emergency operation cited by Mr. Goldmark at Sault Ste. Marie,* was not an unqualified success, since it is understood that improvised wood closures had to be made, because some of the leaves had failed to go down. The degree of closure, even then, was not sufficient to admit closing a pair of gates on the leakage current without danger.

The dam at New Orleans was placed in service after remarkably few adjustments. It has been operated once each month, as a matter of routine, since 1923. There is no knowledge of any failure to function. The great weight and strength of the vital parts should be much in its favor in an emergency. The chances for sticking have been reduced to the simplest number of surfaces, namely, two. Jamming, due to "drawer action", has been eliminated by the design. The operation of the New Orleans Dam is more largely automatic than is the case at Panama. The personal factors affecting the emergency operation have been reduced by having only one operator.

The duties of the operator are simply the ordinary duties of a crane man. The principal thing to look out for is to see that, after landing the girders below water, he slacks off on the hoist far enough to allow the unlatching mechanism at both ends to function. There is a possibility that if this is not done, he may have released one end of the girder and not the other. If, under these conditions, the girder is hoisted, it may be swept out of the slots on the lock walls and defeat the purpose of the entire investment. The hoist is so powerful that an inexperienced operator, during routine operation, has some difficulty in sensing whether he has only the sinker casting or the sinker casting plus one end of the girder on a hook. The water has a high turbidity and nothing can be seen below the surface.

The New Orleans dam cannot be operated by hand. It is, therefore, exposed to the danger of an interruption in the power supply. The stop-log girders were shop-fabricated at Pottstown, Pa., and were sent completely assembled to New Orleans by rail.

W. J. Barden,† M. Am. Soc. C. E.—As stated by Mr. Goldmark,‡ the risk of serious accidents to properly constructed and operated locks is in general small, but there is always the inherent danger of a ship striking and injuring the gates with consequent serious damage. Provisions against such accidents are all matters of insurance and, like any other insurance, cost money and

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2619.

† Col., Corps of Engrs., U. S. A., New York, N. Y.

‡ *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2618.

are, therefore, used to such extent only as the circumstances in each case appear to justify.

Emergency dams, as Mr. Goldmark states, are not essentially protective devices, but a means by which damage done by the breakage of lock-gates and the consequent flow of water through the lock may be minimized by stopping the flow more quickly than could otherwise be done. The emergency dam designed by him and constructed at the New Orleans Inner Navigation Canal is a new application, on a much larger scale than had ever before been undertaken, of the simple stop-log method of closing water openings.

At the Lake Washington Ship Canal locks, the condition existed under which emergency dams are indicated as desirable, namely, a large body of water above the lock which would be drained were the flow not stopped in a reasonable time. There, the damage would not be done below the lock, as in the case of New Orleans, but above it. Mr. Goldmark estimates* the discharge that would take place at New Orleans as 70 000 sec-ft. As a comparison that computed for the Lake Washington lock would be about 58 000.

The type of dam adopted for the latter lock was an adaptation of the balanced cantilever turn-table bridge type used at Sault St. Marie, Mich., and at Panama, and designed to decrease the cost at the expense of some speed in placing.† All parts are stored on the lock wall and placed in position by a stiff-leg derrick, first the bridge supporting the upper end of the six wicket girders, next the operating bridge placed up stream and carrying the machinery for lowering the girders after they have been attached to the main bridge, and then the gates, twenty-four in number.

This dam has been placed in slightly more than 4 hours against as much head as could be secured by opening the culvert valves to full capacity. The lowering of the lakes, the area of which is about 26 000 acres, in that or even somewhat greater time, would probably not be sufficient to cause any very material damage.

The lock is 80 ft. wide, with a draft of 36 ft. over the upper miter-sill. The cross-section of the opening is, therefore, a little more than 3 000 sq. ft., as compared with 4 125 sq. ft., for the New Orleans lock. The total cost, the work having been completed about 1924, was approximately $175 000, or somewhat less than one-half that at New Orleans.

At the small lock, which is 30 ft. wide, with about 17 ft. on the upper sill, the emergency dam consists of five rolling girders or "stop-logs" which are lowered into recesses in the lock walls by a hand-operated stiff-leg derrick.

The paper states‡ that the time required for placing the dam at New Orleans is about the same as that at Panama, namely, 1 hour. It would be interesting to know whether this is in still water, and what experience, if any, has been had in placing it under conditions as nearly similar as practicable to those in time of emergency.

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2620.

† For more detailed description of this dam, see the paper by the writer and A. W. Sargent, M. Am. Soc. C. E., entitled "The Lake Washington Ship Canal," *Proceedings*, Am. Soc. C. E., August, 1927, p. 1240, *et seq.*

‡ *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2640.

Roger D. Black*, M. Am. Soc. C. E.—Mr. Goldmark's excellent paper and the discussion which has followed it bring sharply to mind certain extremely interesting conditions surrounding the general subject. One is the paucity of actual experience in the operation of these dams under the situation of extreme emergency which they are designed to meet; another is the grave seriousness of the results in the event that one of them fails to function when the emergency occurs. A third condition is that a service test of an emergency dam must always be difficult and dangerous and is often found to be impracticable. From these three conditions it is apparent that the design must be based almost entirely upon the correct application of sound theory and yet must produce a structure which can be depended upon in the emergency. Obviously, the problem is one of the most serious and difficult which arises in engineering practice.

Mr. Goldmark's paper is particularly interesting in that the dam which he describes is almost unique and a radical departure from the general type which for many years might have been looked upon as almost standard for major locks in the United States—the type which was adopted for the Panama Canal locks and the great lock at Seattle described by Colonel Barden,† and the only type which in a serious practical emergency is known to the speaker to have functioned successfully, namely, in the accident at the Canadian Sault Ste. Marie.

It will be recalled that the stop-log principle, as against the Panama Canal type, was adopted at New Orleans on the score of economy in first cost. As this consideration may lead to its further adoption which, in turn, would undoubtedly follow a careful re-study of all existing emergency dams, it seems manifestly proper to invite attention to another emergency dam—a dam differing materially from either of those mentioned and one which was designed after giving full consideration to the Panama and other then existing types. This is the emergency dam‡ protecting the lock in the concrete dam crossing the Hudson River at Troy, N. Y., which was built by the Federal Government between 1910 and 1915 to replace the old State dam at that point.

The State dam, a stone-filled timber-crib structure, was erected in the early part of the Nineteenth Century, partly for power purposes and partly to create a pool above Troy with adequate depth for the traffic of the Erie and Champlain Canals.

The Federal Government's project of 1910 for the improvement of the Upper Hudson River was undertaken with the particular purpose of providing adequate channel accommodation for the traffic to be expected from the new Barge Canals. Below the State dam at Troy the elevation of the rock underlying the channel bed and the bank conditions were such as to permit open-channel improvement by dredging, rock excavation, and regulation, but the conditions above Troy (including the elevation given the miter-sills of

* Cons. Engr. (Leaycraft & Black, Associated), New York, N. Y.

† See p. 1276.

‡ Described by Frank P. Fifer, M. Am. Soc. C. E., in *Professional Memoirs*, Corps of Engrs., U. S. Army, and Engr., Dept. at Large, Vol. 10, No. 53, September–October, 1918.

the lowest locks of the Barge Canals) were such that, coupled with the possibility and desirability of utilizing power, replacement of the State dam was decided upon.

The transverse dimensions of the new lock conform to Barge Canal standards, that is, a width of 45 ft. with a depth over the miter-sills at the established low water of 14 ft., to ensure under all possible changes in the regimen of the river a navigable depth of 12½ ft. To provide adequately for the combined traffic of the Champlain and the Erie Branches of the Barge Canal System, the lock (Fig. 19) was made approximately one and one-half times the length of the standard Barge Canal locks and given three sets of miter-gates, dividing it into two chambers, one equal in length to the standard of the Barge Canal, the other one-half that length. Using the smaller chamber alone, economy in the use of water is effected in locking the large portion of Barge Canal traffic which moves in boats less than 150 ft. in length. The longer chamber alone will obviously take any craft that the Barge Canal can pass, and with the center gates open the two chambers in combination materially increase the lockage capacity over that of a standard Barge Canal lock. In addition, the culverts and river wall were so designed as to permit the construction in the future of a duplicate lock at a minimum cost.

The head on the dam at low stages, such as 5 000 cu. ft. per sec., is approximately 14 ft., which decreases as the discharge increases until it practically disappears. For example, in the great flood of 1913, with a discharge computed in excess of 200 000 cu. ft. per sec., the slope at the dam was less than that induced by bank contraction at various points in the river below it, the depth on the dam being about 13 ft.

It is obvious that the volume of water and the conditions of flow in the pool above the lock are such as to make it highly desirable to be able to stop flowage through the lock in the event of the destruction of the miter-gates. The importance of having a simple and dependable emergency dam, however, was greatly enhanced by the position of this lock as the gateway to two great canals, and by the responsibility of the Federal Government to limit interruption of their traffic to the absolute minimum.

The Barge Canal System itself is without emergency dams in its locks for the reason that the pool levels and the flow of water to the locks are otherwise subject to control; for example, by the regulating dams along the Mohawk which apply the principle of the Panama Canal type of emergency dam, except that the supporting structures are in the form of fixed bridges instead of swing draws.

The design of the emergency dam adopted for the Troy lock was derived in part by the application of principles used in the movable dams on Western rivers. Economy in first cost and in maintenance was, of course, a consideration; but the governing consideration was dependability in operation, in turn to be derived mainly from simplicity.

The essential elements of the dam are three trestle bents, a service bridge deck, and a set of roller wickets, each wicket being made up of a steel frame about 11 by 5 ft. over all and supporting six buckle-plates. When not in use the trestles nest one on the other in the bottom of the lock chamber (Fig. 20)

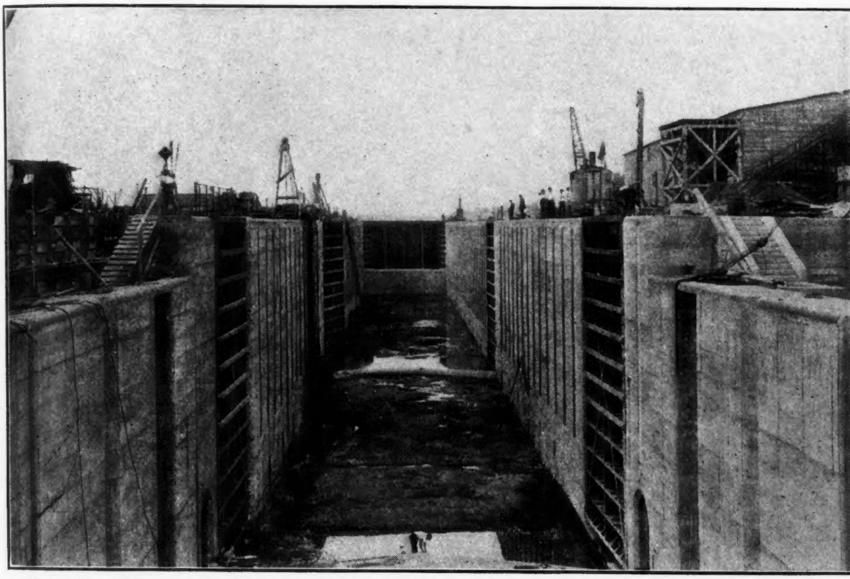

FIG. 19.—TROY LOCK, NEARING COMPLETION, EMERGENCY DAM ERECTED AND IN USE AS COFFER-DAM.

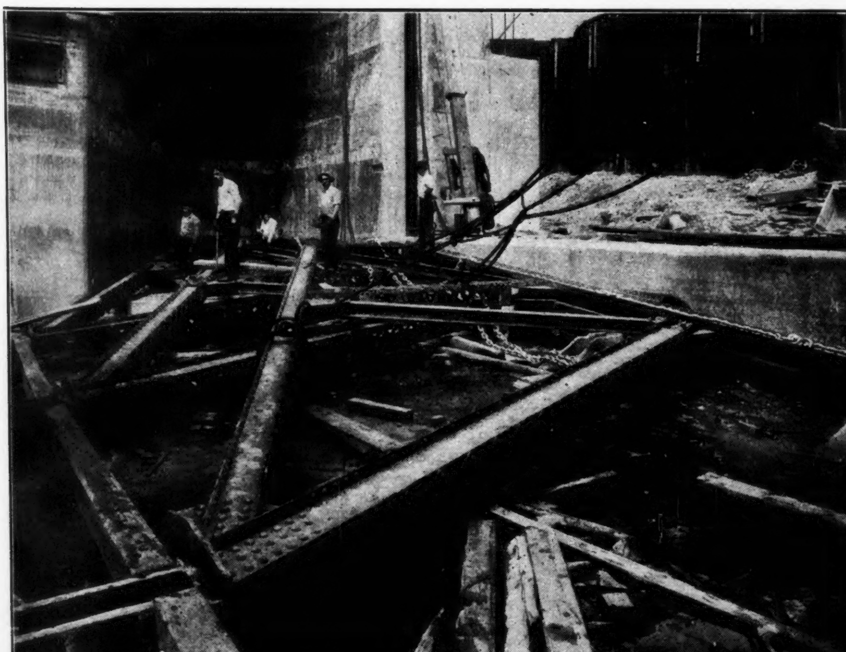

FIG. 20.—EMERGENCY DAM UNDER CONSTRUCTION. TRESTLE BENTS NESTED IN NORMAL POSITION WHEN NOT IN USE.

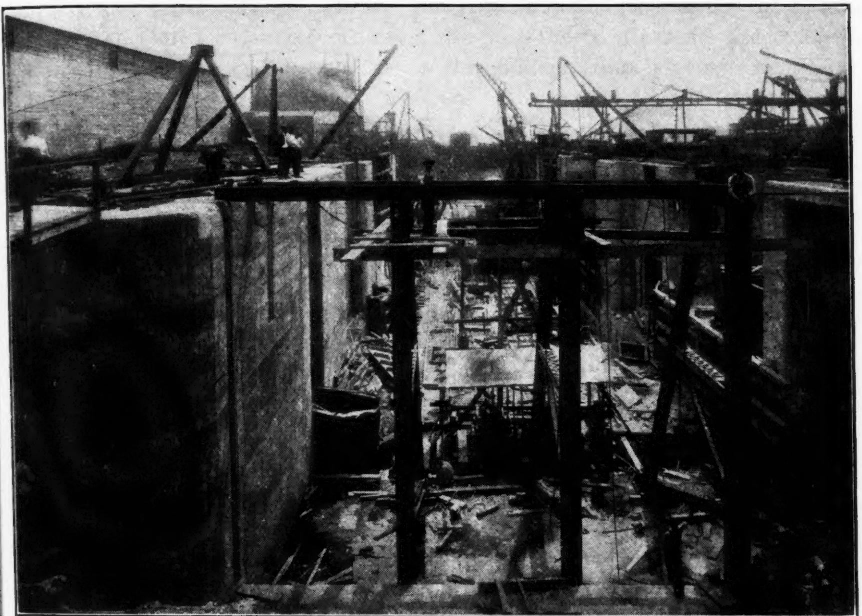

FIG. 21.—EMERGENCY DAM UNDER CONSTRUCTION. TRESTLE BENTS IN SERVICE POSITION AND CONNECTED BY STRINGERS OF SERVICE BRIDGE DECK.

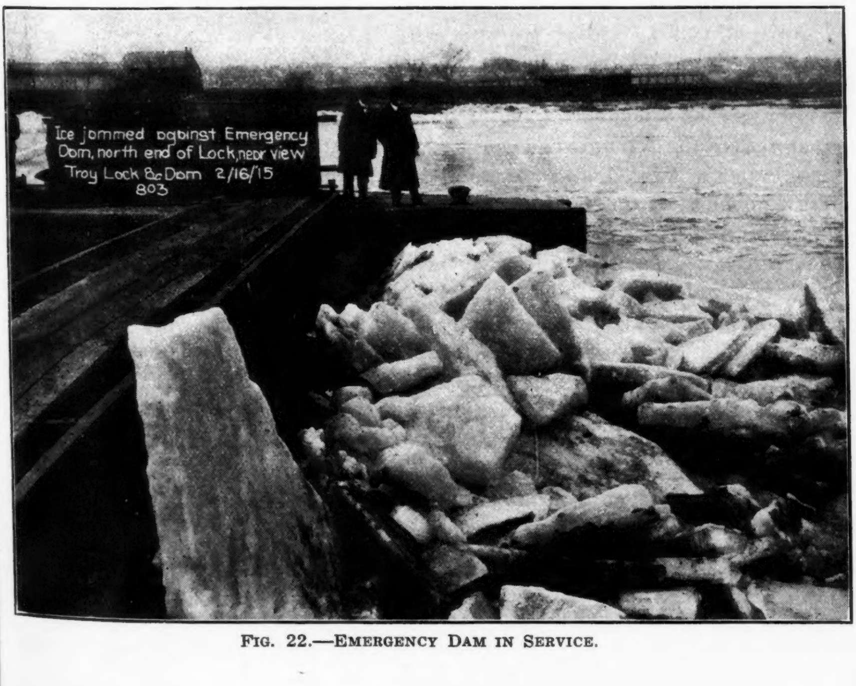

FIG. 22.—EMERGENCY DAM IN SERVICE.

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behind, and protected by, a heavy concrete sill; and the bridge deck and wickets are stored on shore. To erect the dam the trestle bents are revolved 90° in a direction at right angles to the current, thus placing them in a vertical position (Fig. 21); their tops are then secured to each other and to the lock-walls by the service bridge deck, and the wickets are lowered into position one by one in front of and against the trestles, thus shutting off the flow gradually until closure is completed.

The three trestle bents are spaced 11 ft. 3 in. on centers, and the center line of the easterly trestle is the same distance from the face of the land lock-wall. When nested, the upper parts of the bents revolve into a recess in the river wall (Fig. 20). The rollers on the outer ends of the outer sets of wickets bear on steel castings set in recesses in the lock-walls. All intermediate wicket rollers bear directly on the faces of the up-stream trestle legs. The bearing face of the casting in the land wall is in line with the up-stream faces of the trestles, but the similar recess in the river wall (Fig. 22) is advanced up stream sufficiently to clear the trestle recess, so that the westerly set of wickets rests at a slight angle up stream from the others.

The entire structure is designed to resist a hydrostatic head of 26 ft. plus a velocity head of 11 ft. per sec. The three trestle bents, which are identical, are designed to function in the vertical position as cantilevers, transmitting the portion of the load brought to them by the wickets, to anchorages in the lock floor (Fig. 20), through the bearings on which the trestle legs rest and about which they revolve.

Each trestle bent consists of a vertical front, or up-stream, leg. and an inclined back leg, connected by a horizontal member at the top supporting the service bridge deck, and by three inclined and one horizontal web members. There is no member connecting the feet of the trestle legs, but the web members are so arranged (Figs. 20 and 21), as to carry the entire horizontal component of the load to the down-stream bearing. There it is adequately cared for by inclined anchor-rods leading from the shoe of the bearing up stream and downward to a suitable anchorage buried in the lock floor. The up-stream bearing is likewise anchored, but only to resist the upward reaction.

Each up-stream vertical leg is divided by the web members into three panels. In the lowest panel it acts as a cantilever (extending downward) and in the upper panels as a continuous girder, thus carrying the horizontal load to the web members, and relieving the up-stream bearing of all horizontal thrust. The panel points are so spaced that, under this horizontal loading and the tension due to the vertical reactions, a uniform section is used in the front leg with practically uniform unit stresses throughout its length.

The interesting advantage gained in concentrating the entire horizontal thrust at the down-stream bearing, is that all uncertainty is removed as to its distribution between the two bearings under unequal wear or faulty setting of their surfaces, and that the anchorage can be designed accordingly with precision.

The tops of the trestle bents are connected by a heavy chain which, when they are nested, lies on them and leads across the lock to the land wall where

it reaches the surface through a vertical recess. To raise the bents this chain is hauled in by a winch bringing them into a vertical position successively. As they reach the vertical position their tops are connected by light channels which hold them in place and become the stringers on which the service bridge deck is laid. From this service bridge the wickets are lowered by chains and if necessary forced down, shutting off the flow bit by bit. The lowest roller wickets in the erected position rest on the concrete sill previously mentioned. Leakage is controlled by timber stops bolted to the wickets, and can be prevented entirely by simple caulking after the dam is erected.

This emergency dam performed an important function during the construction of the lock. As soon as the up-stream sections of the lock-walls were completed, the dam was erected. This permitted the removal of the northerly end of the steel sheet-pile coffer-dam which was re-erected near the southerly end of the lock. For a period of two years the emergency dam thus acted as a coffer-dam (Fig. 19) and incidentally withstood exceptional pressure from ice and the ravaging effect of several periods of high water. It has since been erected almost annually to permit complete unwatering of the lock and inspection of itself as well as of the upper miter-gates.

In a report* submitted by the Albany Office of the United States Engineer Department on May 14, 1919, the cost of the emergency dam at Troy is given as "cost in place, 93 398 lb. @ \$0.05¾, \$5 360.53".

The width of the lock is 45 ft. and the depth on the miter-sills, 14 ft. This gives a cross-section protected by the dam of 630 sq. ft. at a cost of \$8.50 per sq. ft., a figure which it is interesting to compare with the data given by Mr. Goldmark† for the Panama Canal and the lock at New Orleans.

When in charge of the construction of the lock in 1913-14 the speaker cherished a desire to give this emergency dam a service test under full head. It is believed that one of the best features of the design is the ease and comparative safety with which such a test could be made. Obviously, the dam would be erected in still water with the upper miter-gates closed. The miter-gates could then be opened and the roller wickets of the emergency dam removed one at a time until the desired flow to give a proper service test and all measurements connected therewith had been produced. During this operation precautions could be taken and tested out to make absolutely sure that the gates could be replaced under full flow. It is doubtful whether the Panama Canal type, or the stop-log type at New Orleans, is susceptible of such a practical test with so little real danger.

In considering this problem probably the most serious uncertainty has always been as to whether under service conditions the friction in the roller bearings of the wickets would be so great as to neutralize their effectiveness, but even if this were the case it is believed that little difficulty would be had in forcing these wickets down with jacks operating from the service bridge; or, if necessary, from a heavy girder placed across the top of the bridge to take the vertical thrust.

* Loaned by courtesy of Col. R. R. Ralston, Corps of Engrs., U. S. Army, Dist. Engr., 1st N. Y. Dist., U. S. Engr. Dept.

† *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2617.

Whatever may be said of other designs it is true that the emergency dam in the lock at Troy was designed and erected with the primary object of dependability in operation, but nevertheless the suggestion is made that its effectiveness can never be completely proved until it is tried under emergency conditions. In view of the importance of this lock and the waterways to which it is an outlet, would it not be interesting for the Society to arrange with the Federal Government for a service test at no distant date?

SAMUEL McC. YOUNG,* M. AM. SOC. C. E. (by letter).†—In reading this interesting paper, one is impressed with the simplicity and the economy of the design. At the same time, an examination of the diagrams for the sinker castings (Fig. 15‡) and the automatic latch mechanism (Fig. 16§) will reveal the results of careful thought and considerable ingenuity.

This equipment has been operated, for testing only, about once a month since its installation in 1922, and no serious difficulty has developed in this respect. However, due in part to local conditions, certain little operating kinks, here and there, have been discovered as a result of experience. For example, an accumulation of silt on the floor of the lock when the Emergency Dam is being placed may interfere with the proper operation of the spring device which should release the hooks. In that case, it becomes necessary to use the hand-operated mechanism provided for releasing the hooks when placing the girders in the storage yard.

Another detail, which may not have been thought of in the preparation of plans, but which is more or less important, is the use of tell-tale ropes or cords for determining when both hooks have become engaged or released, as the case may be. When an Emergency Dam girder is being placed in, or is being removed from, the dam, suitable weights, with ropes attached, are placed one on each end of the girder and two men, standing on the lock-walls opposite the ends of the girder, hold these ropes with their hands, keeping them taut. Then, should only one hook be engaged when the lifting mechanism is being operated, one rope will remain taut while the other will begin to be slack. This condition will indicate that only one hook is engaged and that only one end of the girder is being raised. The lifting mechanism, which is being handled very carefully at this stage of the operation, is stopped at once, or else the girder might become jammed between the lock-walls. It is then necessary to go through the process again of engaging the loose hook or of disengaging the other, depending on what is being done with the girder. It may be more convenient, when lowering the girders into the lock chamber, to attach the tell-tale ropes directly to the girders in a manner in which they may be easily released after the hooks are disengaged.

A few minor modifications of the original design were made in the field. These had to do principally with the latching mechanism and the sinker castings. In order to detach the hooks when the girders are to be floated to the north end of the lock, it is necessary to take the weight of the sinker castings

* Chf. Engr., Board of Commrs., Port of New Orleans, La.

† Received by the Secretary, February 14, 1928.

‡ *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2638.

§ *Loc. cit.*, p. 2639.

off the girder, as the girders have not sufficient buoyancy to carry the sinkers. This is accomplished by inserting in the oblong holes of the castings 2 by 2 by 6-in. steel blocks which transfer the weight of the sinker castings to the axles of the movable sheaves.

Even the weight of the hooks is sufficient to overcome the buoyancy of a floating girder, and, consequently, in order to re-engage the hooks, it is necessary to hold them back with the dogs to permit the latching mechanism to be lowered to a position where the hooks will engage. In this position, the dogs are tripped and the hooks take hold. To facilitate the proper alignment of the sinker castings and latching devices in picking up a girder, two cone-shaped steel dowels, 4½ in. in diameter by 2 in. high, have been attached to the bottom of each sinker casting, and corresponding holes were drilled in the tops of the girders. To prevent these castings from swinging about their supports, suitable guides and braces have been attached to the crane structure. The counterweights, which were attached to the hooks, were found to be unnecessary, and their removal has facilitated the detaching of the hooks.

While this Emergency Dam has not been tested under the actual conditions for which it was designed, there is every reason to believe that it would be equal to such a test.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

METHODS USED IN THE CONSTRUCTION OF TWELVE PRE-CAST CONCRETE SEGMENTS FOR THE ALAMEDA COUNTY, CALIFORNIA, ESTUARY SUBWAY

Discussion*

By Joseph A. Kitts, Assoc. M. Am. Soc. C. E.

Joseph A. Kitts,† Assoc. M. Am. Soc. C. E. (by letter).‡—This paper is a valuable record of the methods of pre-casting large tube sections, as viewed by the engineer who was Superintendent of the work for the Contractor, and he (the author) deserves commendation for the thorough manner in which the details of equipment and operations are presented. The writer observed the construction of the twelve segments as Concrete Technologist for Alameda County and will discuss the subject of the paper from that point of view.

The cylindrical-shaped, closely reinforced shell required concrete of good workability, flowability, and cohesion in the fresh mix; the final location of the structure in sea water demanded that the concrete be dense, impermeable, and, of course, of ample strength.

With due consideration of these conditions and requirements, ample cement was specified by the engineer and provisions were made for controlling the testing, proportioning, mixing, and placing of the concrete.

For Class "A" concrete, 2 bbl. of cement per cu. yd. in place and a minimum compressive strength of 2 500 lb. per sq. in., at 28 days, were required. For Class "B" concrete, 1½ bbl. of cement and a compressive strength of 2 000 lb. per sq. in. were required. The contractor was held responsible for results and the workability of the concrete had to be satisfactory to him.

* This discussion (of the paper by Alvin A. Horwege, Assoc. M. Am. Soc. C. E., published in December, 1927, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Concrete Technologist (Kitts & Tuthill), San Francisco, Calif.

‡ Received by the Secretary, January 27, 1928.

The aims in the control of the concrete production were, to maintain: (*a*) The required strength; (*b*) the essential workability, flowability, and cohesion in the fresh mix; (*c*) the highest possible density and impermeability under the conditions; and, (*d*) a uniform concrete with a minimum of honeycomb seams between pours, and shrinkage cracks.

New developments in methods of aggregate and concrete testing, and of proportioning, and production control were used. These comprise:

(1) A comprehensive physico-mathematics of concrete materials and mixtures.
(2) Co-ordination of the tests for the various physical characteristics of aggregates, and co-ordination of aggregate, aggregate mixture, and concrete mixture tests.
(3) Co-ordination of field and laboratory measurements by actual, dry-rodded, loose-measured, and inundated volume, and by weight.
(4) An algebraic method of combining any number of sizes of aggregates for uniform grading of any coarseness modulus, proportioning by size, and absolute volume of particles.
(5) Co-ordination of the fundamentals of the various theories of proportioning concrete mixtures.
(6) Establishment of constant technological control as a daily routine of the concrete manufacture.

These are comprehensive subjects and only a general outline of the aggregate testing and aggregate and concrete proportioning methods will be given.

A concrete laboratory was maintained at Hunters Point. Preliminary research tests were made of the aggregates and of aggregate, mortar, and concrete mixtures, varying the grading, fineness modulus, and consistency for two cement contents (1.5 and 2.0 bbl.) to determine the strength, density, water-cement ratio, weight, cement-space ratio, filler-voids relations, etc. One man-hour of laboratory control was maintained for 10 cu. yd. of concrete. Routine tests of aggregates as received, were made to determine specific gravity, density, fineness modulus, grading, moisture content, bulking, absorption, and content of silt, organic matter, and inferior particles. Calculation and re-adjustment of proportions of aggregates and water were made as required by the natural variation in the grading, density, moisture, absorption, and bulking of the individual aggregates as received. A laboratory duplication was made of each change of proportions to check the physical characteristics expected for that grading, cement, and water content. Routine slump, wash, and compression tests were made for each pour.

Aggregate Test Methods.—The new features of the procedure of testing aggregates are the "weight-volumetric" method of measurement, the regular use of a full container, and the co-ordination of measurements. The procedure is as follows:

(*a*) Weigh the water, filling container.
(*b*) Weigh the loose, moist aggregate, filling container.
(*c*) Dry the aggregate and weigh it.
(*d*) Make a standard sieve analysis.
(*e*) Make a standard silt test.
(*f*) Make a standard test for organic matter.

(g) Weigh the dry-rodded aggregate, filling container.
(h) Weigh the dry-rodded aggregate, inundated.
(i) Allow this to stand 3 hours, then drain, blot, and weigh it.

Having determined these values by measurement, the following may be computed:

(j) The apparent density of the dry-rodded aggregate $= \frac{(a) - (h) + (g)}{(a)}$.

(k) Apparent specific gravity of the aggregate particles $= \frac{(g)}{(j) \times a}$.

(l) Apparent density of loose, moist aggregate $= \frac{(c) \times (j)}{(g)}$.

(m) Weight of moisture to weight of dry aggregate $= \frac{(b) - (c)}{(c)}$.

(n) Bulking of dry-rodded volume by moisture and loose measurement $= \frac{(g)}{(c)}$.

(o) Volume of moisture to dry-rodded volume $= \frac{(g) \times [(b) - (c)]}{(c) \times (a)}$.

(p) Weight of dry-rodded aggregate per cubic foot $= 62.4 \times (k) \times (j)$.

(q) Weight of water absorbed to weight of dry aggregate used $= \frac{(i) - (g)}{(g)}$.

(s) Volume of water absorbed by apparent volume of aggregate $= \frac{(k) \times [(i) - (g)]}{(g)}$.

(t) Relation of proportion of moisture by volume of dry-rodded aggregate to proportion by weight in the corresponding loose, moist aggregate $= \frac{(o)}{(m)} = \frac{(a)}{(g)}$.

Measurement (a) provides a check for the standard container and also makes it possible to use a bucket or other suitable vessel of unknown volume in case of need.

TABLE 2.—Typical Characteristics of Sand and Rock.

Factors.	Sand.	Rock.
Apparent density (dry-rodded)	0.663	0.576
Apparent specific gravity	2.654	2.695
Moisture content (by absolute volume)*	0.136	0.014
Absorption (by absolute volume)	0.04	0.025
Bulking of dry-rodded volume	1.245	1.069
Fineness modulus	3.136	7.31
Discolored rock (by absolute volume)	......	0.094
Inferior rock (by absolute volume)	......	0.014

* "Absolute volume," as used herein, denotes "apparent volume," as generally termed by concrete physicists. It is the volume within the surfaces of the particles in the cases of aggregates and cement, and the volume at normal temperature in the case of water.

Results of Aggregate Tests.—A fine and coarse sand were tested before each delivery, by assistants stationed at the point of origin, and pre-mixed by the dealer in proportions determined by the assistants. Two sizes of rock

were pre-mixed in the same manner. The materials varied somewhat in physical character with each delivery. Typical physical characteristics of the sand and rock, as received at Hunters Point, were as given in Table 2.

Typical gradings of the aggregates, as received, were as shown in Table 3.

TABLE 3.—TYPICAL CONCRETE AGGREGATE.

Sieve No.	Percentage Retained.	
	Sand.	Rock.
100	97.9	100.0
48	82.6	99.9
28	54.7	99.8
14	43.2	99.6
8	29.2	99.5
4	6.0	97.6
⅜ in.	0.0	86.0
¾ in.	....	47.6
1½ in.	....	0.0
Fineness modulus............	3.14	7.30

New Method of Grading Aggregates.—An algebraic method of grading three or more sizes of aggregates for practical uniformity of any fineness modulus, as used on this work, should be of considerable interest to those familiar with the graphical method of determining proportions for uniform grading as developed by the late William Barnard Fuller, M. Am. Soc. C. E.*

The new method combines the fineness modulus principle with the grading equation, $r = 1 - \left(\frac{d}{D}\right)^n$, in which, r is the proportion (by absolute volume) retained by a given screen opening of d in., D is the maximum size of aggregate, and n is an exponent. A grading of any desired fineness modulus is possible by controlling the value of n. The practical values of n vary from 0.45 to 0.60, increasing with the cement content and fineness modulus. When n is 0.5, the curve is a parabola corresponding to Fuller's theoretical grading. From this curve, he deducted a weight of sand equal to the weight of cement used, thus increasing the fineness modulus with the cement content, but losing a measure of uniformity.

The writer's method is to select the maximum desirable fineness modulus consistent with character of aggregate, cement content, and workability, and then find the corresponding value of n.

The desirable fineness modulus for Class "A" concrete was found in the preliminary tests to be 6.00. The grading equation for fineness modulus = 5.99 and $D = 1\frac{1}{2}$ in., is,

$$r = 1 - \left(\frac{d}{1.5}\right)^{0.56}$$

and the grading is as listed in Table 4.

* "The Laws of Proportioning Concrete," *Transactions*, Am. Soc. C. E., Vol. LIX (December, 1907), p. 67.

TABLE 4.—MATERIAL GRADED FOR CLASS "A" CONCRETE.

Sieve No.	Square opening, inches in diameter.	PROPORTIONS RETAINED BY ABSOLUTE VOLUME.	
		Mixed aggregate, r.	Sand, $\frac{(r-r')}{(1-r')}$.
100	0.0059	0.9550	0.8556
50	0.0117	0.9340	0.7882
30	0.0234	0.9032	0.6893
16	0.0469	0.8564	0.5392
8	0.0937	0.7884	0.3209
(4)	0.187	0.6884*	0.0000
⅜ in.	0.375	0.5399	......
¾ in.	0.75	0.3217	......
1½ in.	1.5	0.0000	......
Fineness modulus......	....	5.99	3.19

* The value of r' is 0.6884 and is the proportion of coarse aggregate.

The fineness modulus of the coarse aggregate is determined by proportion to be,

$$\frac{5.99 - 3.19\,(1 - r')}{r'} = 7.25$$

The fine and the coarse aggregates were accordingly pre-mixed for the ideal fineness moduli of 3.19 and 7.25, respectively, the dealer accomplishing this generally with satisfactory accuracy.

New Basis of Concrete Proportioning.—An (*a*) absolute volume of mixed aggregate composed of a uniform grading of particles (by diameters and absolute volumes as in Table 4) with (*b*) an absolute volume of cement to a unit volume of concrete and (*c*) an absolute volume of mixing water to a unit volume of cement, was used as an exact basis of mixture. For example, a typical basis of mixture for Class "A" concrete (assuming 1 cu. yd. of concrete) is as follows:

Absolute volume of cement.............$2 \times 4 \times 0.485 = 3.88$ cu. ft.
Mixing water ....$2 \times 4 \times 0.80 = 6.40$ " "

Absolute volume of aggregate $\left(\text{graded to } r = 1 - \left(\frac{d}{1.5}\right)^{0.56}\right)$

$$\frac{27}{0.97} - 3.88 \text{ (cement)} - 6.40 \text{ (water)} = 17.56 \text{ `` ``}$$

(The determined yield of concrete in place is 0.97 of the sum of the absolute volumes of aggregate, cement and water).

With aggregates as given in Tables 2 and 3, the absolute volumes of fine and coarse aggregates are determined from the fineness moduli and the total volume of aggregates as follows:

$$17.56 \times \left(\frac{7.3 - 5.99}{7.3 - 3.14}\right) = 5.53 \text{ cu. ft. of sand}$$

and,

$$17.56 - 5.53 = 12.03 \text{ cu. ft. of rock}$$

The mixing water, correcting for moisture content and absorption of the individual aggregates, is:

$$6.40 - 5.53\ (0.136 - 0.04) - 12.03\ (0.014 - 0.025) = 6.00 \text{ cu. ft.}$$

The proportions, measuring aggregates by loose-moist volume, are:

	Loose volume.
Cement	2 bbl.
Added water	6.00 cu. ft.
Sand, $\left(\frac{5.53}{0.663}\right) \times 1.245 =$	10.4 " "
Rock, $\left(\frac{12.03}{0.576}\right) \times 1.069 =$	22.3 " "

It should be kept in mind that these calculated proportions are correct only as long as the characteristics of the aggregates remain as given and that about 1 man-hour of laboratory control for every 10 cu. yd. is necessary to keep up with the natural variation of the aggregates.

The characteristics of the usual Class "A" and Class "B" mixes are given in Table 5.

TABLE 5.—CHARACTERISTICS OF CLASS "A" AND CLASS "B" MIXTURES.

Factors.	Class "A".	Class "B".
Fineness modulus	5.9 to 6.1	5.8 to 6.0
Grading, n	0.54 to 0.58	0.52 to 0.56
Barrels of cement per cubic yard	2.00	1.50
Water-cement ratio	0.80 to 0.84	0.90 to 0.94
True mix, cement to dry-rodded aggregate	1 : 3.07	1 : 4.32
Density	0.83 to 0.87	0.82 to 0.86

The usual proportions for 1 cu. yd. were:

	Class "A".	Class "B".
Cement, in sacks	8	6
Sand, loose moist, in cubic feet	11.4	21.8
Rock, loose moist, in cubic feet	21.8	23.1

Compression Tests.—A sample of four to six compression specimens was taken of each pour. Where six specimens were taken, two were turned over to the contractor for independent testing. The specimens were tested at 2, 3, 7, and 28 days and some at other ages.

In view of the erratic nature of compression tests and the law of probability and error in measurement, the probable strength of the samples was determined by taking the average of the following three determinations:

(1) The 28-day strength as measured;

(2) The 28-day strength indicated by the 7-day strength in accordance with the U. S. Bureau of Standards formula, $S_{28} = S_7 + x \sqrt{S_7}$ (x having been determined as 27.7); and,

(3) The 28-day strength as indicated by any two strengths at ages greater or less than 28 days, as determined by the semi-log age rate formula,

$$S = k\ (\log a) + c$$

in which, S is strength at any age, a, in days, and k and c are constants for the particular sample.

Analyses of fifty samples of Class "A" and twenty-three samples of Class "B" concrete show the results given in Table 6.

TABLE 6.—COMPRESSIVE STRENGTH AT 28 DAYS, IN POUNDS PER SQUARE INCH.

	Class "A", in pounds per square inch.	Class "B", in pounds per square inch.
Minimum	2 965	1 980
Average	3 715	2 725
Maximum	4 415	3 890
90%	3 240 to 4 415	2 240 to 3 390
75%	3 445 " "	2 460 " "
50%	3 775 " "	2 670 " "
25%	3 965 " "	2 950 " "
10%	4 200 " "	3 130 " "

Assuming that the lower 10% of samples is due wholly to the usual adverse errors of compressive tests, the minimum indicated strengths at any age, in any part of the concrete, based on the highest of the lower 10%, are expressed by the following equations:

Class "A",

$$S = 2\,074 \log a + 248$$

Class "B",

$$S = 1\,794 \log a - 356$$

This indicates that the probable minimum strength of the concrete in any part of the shell, at the age of 100 days, is about 4 400 lb. per sq. in. and that in the roadway slab, about 3 200 lb. per sq. in.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE VIRGINIAN RAILWAY ELECTRIFICATION

Discussion*

By Messrs. W. B. Potter, Marcel Japiot, L. K. Sillcox, I. Öfverholm, Ralph Budd, Guido Semenza, F. H. Shepard, E. R. Hill, Norman Litchfield, Sidney Withington, and George J. Ray.

W. B. Potter,† M. Am. Soc. C. E. (by letter).‡—The author has written a very interesting account of an important railway improvement and is to be complimented on his paper and for his contribution to the engineering accomplishment which is described therein.

A substantial increase in tonnage capacity of the Virginian Railway with a material reduction in the number of locomotives and operating expenses is a result which speaks for itself. A comparison of the capital charges and operation under steam and electrification will be awaited with interest. A further comparative estimate based on a steam service equivalent to the heavier electric service might be even more interesting.

The electric system chosen with its constant speed characteristic appears to be well adapted to the particular requirements of this division of the Virginian Railway. It does not follow, however, that this is the only electric system capable of meeting the requirements.

Regeneration, as in this case, would seem to be a very desirable feature for any heavy grade operation. The return of energy thus made possible is in the interest of economy. However, is not the elimination of air-braking, with its consequent wear and tear on the rolling stock, an equal, if not greater reason for using regeneration? By attention to dispatching, the regenerated energy might be expected to reduce the fluctuation in power demand, but, after all, the purpose of a railroad is transportation and in an issue between power demand and train movement, it is likely that the trains would have the right of way.

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* This discussion (of the paper by George Gibbs, M. Am. Soc. C. E., published in January, 1928, *Proceedings*, and presented at the meeting of February 1, 1928), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chf. Engr., Railway Eng. Dept., General Electric Co., Schenectady, N. Y.

‡ Received by the Secretary, February 1, 1928.

Aside from conditions to the contrary, it would seem as if a power company having a diversity factor, and, therefore, a better load factor, should be able to sell power with profit for at least the same price as the cost of producing it in a station having only the railway load. The fixed charges, in particular, would be reduced by the better load factor.

The results obtained from pulverized fuel* with the widely fluctuating demand for steam, appear fully to justify its use. It is an excellent showing for pulverized fuel under these extreme conditions. The diagram (Fig. 7†), showing how uniformly the steam pressure is maintained under the operating requirement, is certainly an excellent record.

Successful current collection may be defined as the absence of sparking between the collector and the contact wire. With a system of overhead suspend wire having the essential "contact quality", it has been fairly demonstrated that several thousand amperes may be taken from a collector without visible sparking. In fact, current collection with more than four times the amperes per collector on the Virginian, has been in successful operation for a number of years. It is widely recognized that sparking between the contact wire and collector has a far more destructive effect than the wear resulting from mechanical friction.

The use of side-rod drive for electric locomotives would seem to be more of a special application, and although well suited to these particular locomotives,‡ it seems probable that the trend of locomotive development will be more toward the use of rotary parts only. The requirements as to the strength of rods and side-frames in the motor-drive locomotive are considerably more severe than for a steam locomotive of corresponding tractive effort. That these electric locomotives have been so successful is to the credit of both those who designed them and those who built them.

The use of locomotives weighing 78 000 lb. per driving axle so far exceeds the general practice that the effect on the track is worthy of careful study. It may possibly be found that this weight, in an electric locomotive with a true running balance, will be comparable in its action on the track with a steam locomotive having less weight, but with an unbalanced rotating factor.

Every railroad electrification has successfully performed the service for which it was intended, and the Virginian is to be congratulated as a successful demonstration of the heaviest service yet undertaken.

MARCEL JAPIOT,§ ESQ. (by letter).||—The writer fears that he may seem to be not fully entitled to discuss a paper coming from such a world wide authority in railway electrification as Mr. Gibbs. Moreover, while he had the pleasure of investigating in detail several years ago the wonderful electrifications of the Pennsylvania Railroad and of the Norfolk and Western Railway, he did not find the opportunity to inspect the Virginian Railway electrification. Nevertheless, it may be of interest to know how the American

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 21.

† *Loc. cit.*, p. 20.

‡ *Loc. cit.*, p. 47.

§ Vice-Chf. of Motive Power, Paris, Lyons & Mediterranean Ry., Paris, France.

|| Received by the Secretary, February 6, 1928.

situation in the matter of main-line electrification may be seen from the other side of the water.

The most striking result of the Virginian Railway electrification is certainly the increased capacity of the line. It nearly doubled the speed of the heaviest trains in the world, on such severe grades as those prevailing in the Allegheny Mountains. That is enough to show that the decision to electrify was a most wise one, even if the direct savings had not paid a substantial return on the investment.

The writer agrees fully that, considering the magnitude of the problem, the single-phase overhead line was the best solution, and even perhaps the only one available at the present stage of development. That conclusion may seem strange, coming from a French engineer who advocates direct current for main-line electrification in his own country; nevertheless, it is very easy to justify it.

No system of electrification may be said to be the best in itself; each problem must be examined separately, and the engineer must look, in each case, for the system best suited to the special conditions involved in that individual problem. That was exactly the position that French engineers took when they selected the direct current at 1 500 volts as standard in France.

However, with such extremely heavy loads as those to be cared for in the Virginian Railway electrification, there is no doubt that it was necessary to raise the voltage on the contact line as much as possible, in order to bring down the amperes to be collected by the electric locomotives. Therefore, the single-phase overhead line, with its much higher voltage, seems to be a necessity in that case. The Virginian Railway, with its heavy trains and its severe grades, was indeed the most appropriate field to be found anywhere for the single-phase system.

If the conditions of the problem are changed, the decision becomes reversed. Such was the case on the Illinois Central Railroad, where the scale turned, as it did in France, in favor of the direct current at 1 500 volts.

While the electrified lines in the United States are more important than anywhere in the world, they include only a small part of its vast system of railways. Moreover, there have been, until now, very few physical connections between electrified trunk lines using different types of current, alternating and direct. These two facts seem to explain why American engineers did not feel any urgent need for standardizing on a definite type of current for main-line electrification, as did several European countries.

With the unrestricted liberty still prevailing in the American field, engineers in the United States have the opportunity to give to each individual problem the most favorable solution. On the other hand, standardization must always be a compromise between the advantages and the drawbacks of the different systems of electrification for several various problems.

The necessity of such a compromise does not seem to be imminent in the vast territories of the United States. Nevertheless, it appears from the recent developments that, if any compromise had to be made at an early date (which does not seem to be probable), the single-phase overhead line might be adopted successfully in America.

That does not mean that the writer is ready to depreciate in any way the advantages of the direct current for many definite problems of electrification; but nobody can deny that the much higher voltage now allowed on the contact line by the single-phase alternating current is of the utmost importance for the average conditions prevailing on the American railways.

Moreover, the flexibility of that system for electric motive power must be kept in mind, because it permits the use of either (1) the straight single-phase system, as on the Pennsylvania Railroad and the New York, New Haven, and Hartford; (2) the so-called "split-phase" system (single-phase to three-phase), as on the Norfolk and Western Railway and on the Virginian Railway; or (3), the motor-generator engine (single-phase to direct-current), as on the Detroit, Toledo, and Ironton Railroad, and on the Great Northern Railway. The motor-generator type may perhaps be further improved in the future by using some kind of static or rotating rectifier, instead of the somewhat cumbersome motor-generator set. In any case, the diversity of the solutions available for electric motive power with the same single-phase contact line gives to engineers great facilities to adapt that system to very different conditions, which is a most important quality for a compromise.

Nevertheless, from an engineering point of view, it is fortunate that the necessity for such a compromise has not yet become urgent in the United States, as it gives to American engineers an unlimited field of possibilities for other wonderful achievements, such as that accomplished so successfully by Mr. Gibbs in the Virginian Railway electrification.

L. K. Sillcox,* M. Am. Soc. C. E. (by letter).†—One of the greatest disadvantages of the past for those endeavoring to present a proposition favoring electrification of an existing steam-operated railroad has been that their proposals were based on theory which made it possible of attack by contending that the theory would not be borne out in practice. Mr. Gibbs has been able to avoid such a situation by reason of his long, valuable, and successful experience in connection with the electrification projects carried out on the Pennsylvania, Long Island, and Norfolk and Western Systems, as well as a number of other well known and important undertakings. His contribution to the art of electric railroading has always been worth while, and this paper covers the case accurately and fully.

About 1926, the writer was privileged to make a survey of this line and, therefore, can present some first-hand impressions gained at that time. In the first place this railway has a capacity for dumping 11 400 tons of coal per hour at the seaboard, which is equal to 273 600 tons per 24-hour day. It has always been recognized as a leader in the movement of heavy-tonnage trains, and in the operation of most powerful modern motive power. In May, 1921, all records of heavy trains handled were exceeded when there was a test run on the Third District, consisting of a train of 100 loaded cars, each with a capacity of 120 tons, aggregating 16 000 gross tons. To give some idea of

* Asst. to Pres., The New York Air Brake Co., Watertown, N. Y.

†Received by the Secretary, February 7, 1928.

the rapid expansion of the road's business, it is interesting to observe that, in 1911, a coal tonnage of 2 141 009 was handled; in 1920, this figure was increased to 7 621 555 tons; in 1925, it went to 8 250 000 tons; while in 1926 the total was 11 500 000 tons.

It is well, at the outset, to take seriously the author's remark* that "it is obvious, however, that uniformity [in energy] demand cannot be secured, even approximately, in practical railroading", having due regard to the factors producing reasonable plant cost and economy of operation. When there is sufficient tonnage available, one of the principal features which limits the earning capacity of any railway system is the total amount of tonnage that can be handled over the line in a given time. The project under discussion was undertaken only after most careful consideration of all the comparable methods of successfully handling and expanding competitive traffic demand, and its greatest significance is an economic one.

The electrification of the Virginian Railway has, in general, enabled a much better use to be made of the capital expended in the construction and development of the property up to that time than would have been attainable under any other means involving an equal expenditure. This is mainly due to the more prompt movement of tonnage and greater utilization of the property as a unit than formerly was possible. Under electrified traction the section from the Elmore, W. Va., Yard to Roanoke, Va., a distance of 132 miles, is operated as a one-locomotive district. Under steam operation this was a two-engine district with turn-around runs between Princeton, W. Va., and Elmore. Trains of 6 000 tons are being handled from Elmore to Clark's Gap, W. Va., with one leading locomotive and one pusher behind, each composed of three electrical units. Under steam operation, trains of 5 500 tons were hauled by three Mallet engines, one in front, of the 2-8-8-2 type with a tractive effort of 101 300 lb., working compound, and two pusher engines of the 2-10-10-2 type with a tractive effort of 147 200 lb., working compound. The speed under electric operation is 14 miles per hour, while under steam it did not exceed 7 miles per hour. At Clark's Gap the load of the trains was increased to 9 000 tons, which was taken to Princeton with one locomotive. From Princeton to Roanoke, the train was assisted by a helper, from Whitethorne to Merrimac, Va., and also out of the Princeton Yard. Under electric operation this helper service is eliminated. The speed of the trains varies between 14 and 28 miles per hour, with a consequence that this line, which is of single-track construction (except from Mullens, W. Va., to Clark's Gap), is equipped for a handling capacity of 12 500 000 tons per year, and when this figure is reached the tonnage is to be carried by forty-eight electric units, or sixteen 3-unit locomotives. This same service, if it were handled by the existing design of steam power all of which was of modern construction, would require approximately twenty of the large 2-10-10-2 type Mallets, and forty-four of the 2-8-8-2 type Mallets, a total of sixty-four steam units as against sixteen electrical units; and, in addition, under steam operation, extra helper engines to assist trains out of the Princeton Yard and between Whitethorne and Merrimac would be required. The regenerative system of braking on grades, especially in the case

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 13.

of tonnage trains, is also of great value from a practical railroad-operating standpoint.

The electric locomotives have a 2-8-2 wheel arrangement, with a weight per unit of 212½ tons, so that the weight of the 3-unit road locomotive is 637½ tons. The maximum tractive effort of each unit, as limited by electrical capacity, is 92 500 lb., or 277 500 lb. for the 3-unit locomotive, which occurs at an adhesion of slightly less than 30 per cent. Computing the conservative starting adhesion at 25%, the tractive effort is approximately 77 000 lb., or 231 000 lb. for the 3-unit locomotive. This represents a starting force 57% greater than the compound rating of the 2-10-10-2 type Mallet steam locomotive (one of the largest steam engines ever built) and 31% greater than this engine when working simple. The continuous rating of the 3-unit locomotive is 135 000 lb. at 14 miles per hour and 78 800 lb. at 28 miles per hour. In the high-speed connection of the motors the three units exert 6 000 h.p. continuously. These engines operate on 2% grades and through a maximum line curvature of 12 degrees.

One feature of the project which is of unusual interest and needs to be especially commended has regard to the satisfactory location and thoughtful study given the design and equipment of the shop facilities at Mullens, which will enable the motive power to be kept in proper condition at a minimum of expense and to give a maximum of reliable movement. The greatest need under electrification, as compared to steam operation, is that of comprehensive inspection, since that requires much the greater part of the time of running maintenance forces; whereas with steam motive power, defects are relatively self-evident upon arrival at the terminal, or during the running inspection, and it is the time to make repairs which predominates and not that of inspection. For this reason, it is most unwise for any steam railroad, which may contemplate electrification, to attempt any lack of due consideration, required to maintain electric motive power efficiently, because the two types of operation do not lend themselves to similar treatment in this respect, and the facilities must be set up with a view to the peculiarities of the requirements in each case. The ideal of attainment in transportation is to have all trains run at the same speed, and although that ideal is not fully realizable, it can be more nearly attained with electric than with steam traction.

I. Öfverholm,* Esq. (by letter).†—The writer would like to add some remarks to this very interesting paper.

One of the reasons given for the 88 000-volt transmission line and the seven substations is to make it possible to avoid the stub-end feed and its bad effects on the communication circuits along the railroad. As far as the writer is informed, the result of these arrangements is not fully satisfactory. It seems that, in any case, it has been necessary to make the telephone and telegraph circuits immune from the residual disturbances in these wires.

The writer, therefore, thinks that it would have been better to provide some other means in order to get a more desirable location for the communication

* Chf. Engr., Swedish Govt. Rys., Stockholm, Sweden.

† Received by the Secretary, February 8, 1928.

circuits. According to Swedish practice, the transmission lines and the substations would not have been designed for 88 000 volts. Swedish engineers would have used a trolley line with a potential of 16 000 volts and a balancing wire such as that used on the Virginian Railway, with a potential of 32 000 volts. Thus, they would have had a transmission potential of 48 000 volts, which seems to be great enough for the transmission along the electrified part of the Virginian Railroad.

The trolley wire and its catenary, according to current practice in Sweden, would have been made of copper with a section of 300 000 circular mils for both together. The balancing wire would have been made also of copper and with the same section, and then a feeder line for 48 000 volts with two wires each with a section of 300 000 circular mils would probably have been used. This causes a very severe stub-end feed, which is not to be feared with the precautions now used.

A fair comparison may be made with the most recently electrified line from Stockholm to Göteborg, where very satisfactory experiences were obtained with similar arrangements. On this line, the communication cable belonging to the railroad (which cable is placed in the roadbed), has a disturbance potential, under normal service, running up to a maximum of 10 to 15 volts, and at most severe short-circuits, 47 volts. For this reason there are no disturbances in telephone and telegraph circuits even at short-circuit. The telegraph circuits can be, and are, operated with a ground return on a single wire in the cable.

For such a case as the Virginian, Swedish engineers would have used feeder booster transformers inserted in the trolley wires and the balancing wire midway between the balancing transformers, which would have been spaced at distances of about 6 miles. The arrangement of the transmission would in this way have been simplified very much, and the cost would have been reduced. Also the residual disturbance voltages on the communication wires would not have amounted to such values as now are reached on the Virginian Railway. However, these arrangements might give a somewhat greater potential drop in the contact lines.

Ralph Budd,* M. Am. Soc. C. E. (by letter).†—The Virginian Railway electrification is a good example of traffic and operating conditions which justify electrification purely for reasons of economy. The expedited movement of freight, the saving in fuel and in wages of trainmen and enginemen, and the avoidance of expensive additional tracks for increasing the capacity of the railway combine to make current economies which represent a large return on the investment. Undoubtedly, there are more instances in the United States where electrification of heavy traffic sections of steam railways is justifiable for similar reasons, and like action, therefore, may be expected to follow. On most of the railway mileage of the country, however, the traffic density is not great enough to pay a return on the cost of electrification sufficient to make the investment attractive.

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* Pres., G. N. Ry., St. Paul, Minn.

† Received by the Secretary, February 20, 1928.

The improved efficiency of the steam locomotive makes it more difficult to show great economy in electric operation, as compared with steam, than it was ten years ago. Of course, any study to be convincing to a competent and careful railway board of directors, must compare the estimated cost of operation after electrification with the cost of operation using the most modern and best adapted steam locomotives for the duty imposed, rather than the steam locomotive performance some years ago, or even that of the present. This is a point of some importance because, pending the final solution of a transportation problem, even the best railway company may wisely hesitate about replacing its old steam locomotives with strictly modern ones.

As Mr. Gibbs has stated,* the Norfolk and Western electrification is so closely related to the Virginian, both in location and in operating requirements, that the very successful record of the former naturally influenced the adoption of the same system on the Virginian, thus giving convenience in detouring or the exchange of motive power, if desirable.

Whether or not rod-type electric locomotives are the most desirable does not seem to be demonstrated by anything in this electrification, nor indeed is there anything to show whether locomotive performance, and especially locomotive upkeep, is more favorable with this particular system of electrification than it would be with others. The limited speed control does not seem to be any particular handicap here as it might be under other and more varied conditions. The most striking differences the writer has seen in comparative data on heavy-duty electric railway operation have been in the cost of maintaining the motors, the best showing being made by direct-current motors. That item, of course, is only one, although a very important one, of many which go to make the total of operating cost. In any event, the Virginian Railway undoubtedly has a highly efficient and economical electrification. The engineers who are responsible for designing and directing the work are to be congratulated upon its entire success, and thanks are due Mr. Gibbs for the splendid paper describing the project.

GUIDO SEMENZA,† ESQ. (by letter).‡—The author is correct when he states that the electrified section of the Virginian Railway furnishes an example of the heaviest electric traction development to be found anywhere in the world. This is true for the amount of traffic hauled, for the magnitude of the locomotives, and for the power required by each unit. These features are of such importance as to make this paper one of greatest interest, and the results obtained, both technically as well as economically, are so satisfactory that Mr. Gibbs and his collaborators are to be complimented.

In this clear exposition the author shows that the problems to be solved were quite unique—a very difficult profile of the line, with heavy grades; a traffic of such a character as to be called a "shuttle service"; a small number of very heavy trains, running at a relatively low speed; and a uni-directional movement of the charge. Therefore, if the methods, systems, and devices

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 5.

† Engr., Milan, Italy.

‡ Received by the Secretary, February 23, 1928.

used in solving the problems of this large project appear to be justified and appropriate, one must be careful in extending them to other cases, in which the conditions of traffic are quite different.

The power of 11 000 kw. required by each train justifies the alternating high tension in the contact wire and, although it might at first seem a useless complication to transmit single-phase current when the motors are three-phase, one can only approve of having done away with the serious construction complications and the severe and costly operation of a 10 000-volt, three-phase contact line.

To one who has had occasion to follow closely a rather extensive system of three-phase traction, it seems rather queer that the three-phase motor was chosen in order to meet the severe starting conditions. The three-phase motor has many good qualities, but its drawback has always been considered to be in its starting characteristics, namely, a limited torque, that cannot be improved by slowing down, and that is highly influenced by the line voltage drop.

It is only when the starting is difficult and when the track conditions are bad, that an abnormally high torque is required, with a corresponding increase of current, and the voltage drop then is maximum. Moreover, the starting conditions are likely to be altered by the heating of the water rheostats.

If, in spite of these structural features of the three-phase motor (and it does not appear that the phase converter can improve matters very much), the operation is satisfactory, that means that the dimensions of the motors and rheostates have been calculated so as to meet these special conditions. It would be interesting, however, to compare (as to weights, costs, and efficiencies) the actual use of three-phase to that of direct-current motors in these same locomotives.

F. H. Shepard,* Esq. (by letter).†—The Society is to be congratulated on Mr. Gibbs' admirable paper which shows, in a comprehensive way and in valuable detail, this important and successful railway electrification.

The institution of this undertaking reached its final stages in November, 1922, with the request from the Railroad Company for a tender upon and a recommended plan and method for electrification. This was required in the form of (1) a competitive bid for the apparatus and equipment involved (as manufactured by the Westinghouse Electric and Manufacturing Company); (2) an estimate on the other items going to capital expenditure; and (3) an estimate of cost of operation and savings. The contract was executed under date of April 18, 1923.

Subsequent to the signing of the contract, Mr. Gibbs and E. R. Hill, M. Am. Soc. C. E., took over the execution of the field work, the design and construction for the power house, substation, transmission, and conducting systems, all to suit the requirements for traffic movement as specified in the plan of operation.

A most gratifying part of the whole undertaking has been the confirmation of operating results and advantages which were predicted for this electric

* Director of Heavy Traction, Westinghouse Elec. & Mfg. Co., New York, N. Y.

† Received by the Secretary, February 27, 1928.

operation, all of which were realized with ample margin. The engineering and construction work undertaken by the consulting engineers was planned and executed with smoothness and despatch and in co-ordination with the regular operation of the road under steam.

From the initial run of the first train until the whole section was under complete electrical operation, the addition of electrical units was attended with a distinct and regular improvement in the movement of traffic. The members of the Operating Staff of the Railway Company were alert and responsive in their desire to learn the details of electric operation and had so far mastered the handling of the new motive power that little or no assignment, apart from the road's regular forces, was necessary for instruction of the forces required to operate and maintain the electric locomotives in this unprecedented train service.

As this operation now stands there has been realized an increase in capacity for moving traffic materially in excess of that claimed for the undertaking. Furthermore, the locomotives have capacity for service materially in excess of that required. They can make the run between Clarks Gap and Roanoke in approximately 1½ hours less time than that stipulated; they can handle the specified tonnage up Clarks Gap Hill even if the grade were 5 miles longer; they have demonstrated a momentary capacity for initial start equivalent to moving 6 000 tons on a 2% grade with a single locomotive; and, in addition, they have demonstrated the entire success of close regulation of tractive effort for these exceedingly heavy trains under all conditions of starting and running.

E. R. Hill,* M. Am. Soc. C. E.—The Virginian Railway, built in 1905-07, was planned and constructed in a far-sighted and broad-gauged manner, as a result of which grades were made as favorable as possible considering the region traversed. There were no unnecessary dips or other objectionable features, and only two pusher grades under steam operation and one under electric operation. The bridges were constructed for heavy engine and car loads and thus even before electrification the heaviest trains in the country, and, in fact, in the world, were operated on this line. In view of this and the physical limitations in strength of draft-geared cars in electrifying, it was not thought advisable at the outset to adopt maximum train weights (6 000 tons on Clarks Gap Hill and 9 000 tons elsewhere) materially greater than had been handled under steam operation, although these weights are about 50% greater than could be handled by steam in winter. It will be noted that provisions have been made to increase the trains to 12 000 tons at such time in the future as traffic conditions, construction of cars, and other considerations make this desirable or practicable.

The paper refers to the exacting conditions of the load and the probable difficulty of carrying such a load on a general power supply system. From the nature of the load it would be difficult to carry it on any ordinary power system except a very large one, a much larger system with a heavier power background than was available in that locality at the time.

* Cons. Engr. (Gibbs & Hill), New York, N. Y.

The choice of the split-phase type of locomotive over the motor-generator type or the straight single-phase was somewhat influenced by the facts that there were two railroads in that same valley; that certain combinations of railroads were and are now permissible; and that mutual interests pointed to a uniformity of system and locomotive characteristics as being desirable in that territory. In fact, during the World War, these two roads were operated as one railroad. There were other considerations, of course, which led to the selection of the split-phase type of locomotive which it is believed was the right one under all the circumstances. Furthermore, at that time the motor-generator type of locomotive had not been developed to the point where it could safely be adopted on a large scale.

With regard to the sub-stations, it should be noted that all the power which goes to the trains passes through the main transformers, the balancing transformers being part of the distribution system and affecting the regulation. With this arrangement of transformers, less main transformer capacity is required than with a more frequent spacing of ordinary transformers, partly because of the necessity for having a "spare" at each location. The system was laid out in both ways, and the most economical and efficient one, all things considered, was selected.

The electrification of the line cost approximately $15 000 000—the amount estimated. Allowing credit for the additional facilities that would be required to secure approximately the same capacity by steam, including double-tracking, locomotives, and other facilities, the net additional cost of electrification was about $5 000 000. The purpose of this electrification was to secure increased capacity of the railway, reduce operating costs, avoid expensive and difficult changes in the way of double-tracking, secure smoother, safer, and more reliable movement of trains, and, incidentally, to clarify the air in tunnels, especially the Merrimac Tunnel at the summit of the Allegheny Mountains, which is about a mile in length.

After 1½ years of electric operation it can be said that the expectations have been more than realized in all respects. The direct savings in operating costs affected by the change, as determined from carefully compiled steam-operating costs for this particular division prior to electrification and from actual results since, indicate a decrease in such items of cost per gross ton-mile of more than 50 per cent. This saving on the gross ton-mileage operated last year (1927) was sufficient to pay fixed charges on the gross cost of electrification, or three times that required to pay such charges on the net additional cost of electrification. The operating and the financial results, therefore, have been entirely satisfactory.

NORMAN LITCHFIELD,* ESQ.—The speaker has been closely associated with the Virginian Railway electrification throughout, and the following remarks are more in the nature of emphasis than discussion.

One of the outstanding features of the electrification is the rugged character of the country. Those who are unfamiliar with West Virginia can have little conception of the topography, for while the mountains are not of the

* Mech. Engr., Gibbs & Hill, Cons. Engrs., New York, N. Y.

height and grandeur of other great chains, nevertheless there are a great many of them, placed in such irregular groupings that it almost seems as if Nature had had a great convulsion there. This made the running of the transmission line a difficult matter, particularly in the delivery of materials to the site, new roads having to be cut in many cases.

The climate is more severe than is ordinarily associated in one's mind with railroads in that latitude, making winter work, although generally possible, at times, quite difficult. The "frozen fog" as its name implies is neither ice nor snow, but a sort of frost piled up on the wires, which sometimes attains the diameter of a man's arm. When the sun strikes such a coated wire in one spot, the frost drops off rapidly in the sunshine, but remains on the wire in the shade. This results in an unbalanced condition and the wire whips upward. To take care of this the lower wires are offset from the upper ones, as is shown in Fig. 12.*

The diversified character of the problems encountered in a major electrification is apparent from Mr. Gibbs's paper. Virtually every branch of civil, electrical, and mechanical engineering is involved. One of the most striking features was the necessity for anchoring the power-house foundations to the solid rock to prevent possible dislodgment during river floods. The very patent question might be asked, "Why put the power house in the bed of a river?" The answer is obvious in view of the nature of the surrounding country, that this was the only flat place available, where condensing water could be had.

Some idea of the magnitude of the transportation problem which the Virginian Railway has to meet can be given by a more detailed consideration of the size of the train. Unless one is a railroader it is hard to conceive just what a 6 000 or 9 000-ton train is. As the cars average about 46 ft. in length and weigh (including contents) at an average about 100 tons, a 6 000-ton train contains 60 cars or more and is about 2 800 ft. long. Similarly, a 9 000-ton train has 90 cars and is about 4 100 ft. long, or nearly ¾ mile long. If this were composed of the cars one finds in an ordinary freight train, averaging, say, 50 tons each, it would include about 150 cars and be about 6 000 ft. long. This gives some conception of what a Virginian Railway train really is.

To start a 6 000-ton train on the "hill" may require a tractive force of more than 500 000 lb. with one locomotive at the front of the train and another at the rear, pushing. At present, there are no means of communication between the front and the rear locomotive other than by whistle signals, which often cannot be heard because of intervening hills; hence, it is virtually impossible to start both locomotives at the same time without recourse to an indirect practice. The brakes are applied on the pusher locomotive, and the front locomotive backs the train down into the rear one. A train of this length will have a total slack in the couplings and draft gear of perhaps 50 ft., or more. Hence, the rear engine receives a relatively heavy bump when the slack has been taken up. This is the signal for the rear engineman to apply his power. Even then it is difficult to synchronize with the front locomotive, and some little time may elapse before the rear engine starts moving, in some cases as

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 29.

much as 1 min. This, of course, puts an exceedingly heavy electrical strain on the motor, which is one of the features that makes the induction motor a proper selection for this class of service.

Equally interesting is the action of the locomotive in descending the long 1.5% grade. An ordinary freight train hauled by a steam locomotive, going down grade sways back and forth both sideways and "fore and aft". This is largely due to the fact that it is impossible to keep the brakes applied continuously, because this would burn up the brake-shoes, crack the wheels, and cause havoc generally. With the exception of a few cars on which retainers are set up, it is necessary to release the brakes after the train has slowed down to as low a speed as possible without danger of coming to a full stop, then release the brakes again and let the train gain headway until the maximum safe operating speed is reached. This process is repeated again and again. Although the air-brake has been improved so that it is a marvelous mechanism, it is not perfect, and it takes a relatively long time for its action to be transmitted from the front to the rear of a long train; also, irregular braking action occurs and, in such a case, it is not unusual to break the train in two.

With the electric locomotive, after the train gets about on the crest of the hill, the engineman simply throws a lever, and the motors begin to regenerate, thus acting as a powerful and steady brake, so that the train proceeds down the hill at a practically constant speed, and with little irregular motion. This, of course, gives a very desirable operation, and one which is much easier on the rolling stock.

The electrification is popular with the rank and file of the railroad men (as shown by the expressed desire of the enginemen to obtain the electric runs) and has generally proved its worth.

SIDNEY WITHINGTON,* ESQ.—In presenting as full information as is possible within a brief space, of the large and important undertaking represented in the Virginian Railway electrification, Mr. Gibbs' paper is of great value as a matter of record, and the Society is to be congratulated in having secured this exposition.

One of the outstanding features of the Virginian electrification is the extreme variation of load over a short period of time, in swings amounting to nearly 50 000 kw., as indicated on the load curve in Fig. 4.† Only very large commercial systems would be able to handle such great and extreme variations of load, and the wisdom of choosing pulverized fuel to obtain flexibility at the power plant is obvious. In a plant of this size, with such load characteristics, the consumption of $1\frac{3}{4}$ lb. of coal per kw-hr. net output is a creditable performance.

In connection with the regeneration of power on descending grades, it is of interest to note that the only instances in the United States where such regeneration takes place occur in territories where, on account of the proximity to coal fields or availability of hydro-electric power, the unit cost of energy is relatively low and the money value of the regenerated power is of little

* Elec. Engr., N. Y., N. H. & H. R. R., New Haven, Conn.

† *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 13.

importance. The advantage of regenerative facilities is not so much in the power saved as in the increased safety of operation and ease of handling trains. It is noteworthy that in many instances abroad, regeneration as such has been discarded and replaced by simple electric braking wherein the traction motors operate as generators as in the case with regeneration, but the energy is absorbed in resistance on the locomotive itself and is thus not utilized.

Very much the same idea of balancing transformers was applied in 1913 on the New York, New Haven and Hartford Railroad between New Haven, Conn., and New York, and on the New York, Westchester and Boston Railway, a subsidiary. This type of transformer on the New Haven combines the benefits of 22 000-volt transmission with the balancing effect to reduce interference with telephone circuits. The plan has also been used abroad to some extent.

Poles of H-section were utilized on the New Haven, in 1914, in side-track construction, and, in 1925, on the main-line track between South Norwalk and Danbury, Conn. This type of pole, as Mr. Gibbs suggests,* has great advantage in the use of the flanges in climbing, as the climbers or "skates" devised for the linemen eliminate the necessity for ladders or steps as in the case of tubular poles.

In connection with the electrification of the New Haven Line between South Norwalk and Danbury, a note on the foundation design for the catenary columns may be of interest. The poles or columns were the H-type, of normal sizes, 6, 8, 10 and 12 in., depending on the loads imposed. The foundations were poured in place, and embedded in the concrete of each pier were two channels, set back-to-back and separated by a distance equivalent to the dimension between flange faces of the pole to be set. These channels, and the lower end of the pole, carried holes for 1-in. bolts in sufficient number to develop the required strength of the connection. In the process of setting, the poles were slipped down between the channels and enough bolts installed to hold them, before the work train with the crane moved on to the next pole. In spite of the fact that a considerable amount of both freight and passenger traffic was operated in both directions on this single-track line (which it was necessary to clear sufficiently in advance in order to avoid delays), as many as forty-nine poles were set in a single day in this manner.

Fig. 19† shows the limitation of the inclined type of hanger installed on extremely sharp curves. By laying a straight-edge along the contact wires between pull-off points shown in the photograph, it may be noted that the contact wire assumes nearly the same position which would be the case in chord construction mentioned by Mr. Gibbs,‡ and it has been the experience on the New Haven that for curves sharper than 4°, the chord construction is as advantageous as the inclined hanger type in the matter of length of spans, is somewhat easier to install, and more economical to maintain.

In connection with the design of the catenary system, it is necessary to exercise care to insure that the cost of carefully developed details in the

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 40.
† *Loc. cit.*, p. 41.
‡ *Loc. cit.*, p. 38.

drafting-room, as suggested by Mr. Gibbs,* covering all possible contingencies in the field for each catenary span, is not in excess of that which would be saved by a small amount of additional work done by the field gangs which might be necessary with standardization. It is probable even that some standardization of span details would be justified in actually saving money in the field both in construction and maintenance, by eliminating large numbers of odd lengths of hangers and consequent heavy storehouse expenses in handling material.

Under electric operation the engine-house expenses are noted as being about one-fourth of those accruing under steam operation. This seems a somewhat high ratio unless the Virginian Railway charges more to the account of engine-house expenses than some other railroads. The ratio is usually nearer 1 to 6, or even 1 to 10, on account of the simplicity of electric locomotive terminals. They eliminate the need of coal and ash-handling facilities, and the savings on this account may amount to a considerable sum in the aggregate.

George J. Ray,† M. Am. Soc. C. E.—This paper is a valuable addition to the literature on steam railroad electrification.

The speaker wishes to add a comment as to the regenerating features of operating electric locomotives on heavy grades. The handling of heavy freight trains on steep descending grades is a difficult operation. On the road with which the speaker is connected there is one stretch of 17 miles of $1\frac{1}{2}\%$ grade. On this grade it is necessary to restrict the speed of heavy freight trains to 15 miles per hour in order to avoid "runaways". The customary practice makes it necessary for the engineman to make approximately thirty-five brake applications and releases during the descent; that is, he must apply the brakes, bringing the speed of the train down to about 10 miles per hour, then release them again, and allow the train to run free until it reaches a speed of about 18 miles per hour and then apply them again. With this in mind, the great advantage in the regenerating feature under electrical operation is readily understood.

There has been much discussion about the feasibility of operating trains under automatic train control on such heavy grades. When the brakes are applied at low speed it is necessary to make a full stop before releasing them; otherwise, there might be a catastrophe. On a train of 85 or 100 cars it takes so long for the air to act from the engine to the rear of the train that if an engineer is permitted to release at slow speeds, especially with empties, he is almost certain to buckle the train or pull it in two. It is clear, therefore, that the introduction of an automatic brake application might be a rather serious matter where heavy trains are being handled down a steep grade. There would seem to be less likelihood of trouble with electric traction where the train is operated at a uniform rate of speed such as that indicated by the author. The speaker would like to inquire whether the question of automatic train control was given consideration in connection with either the Virginian or the Norfolk and Western electrification.

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 36.
† Chf. Engr., D. L. & W. R. R., Hoboken, N. J.

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PAPERS AND DISCUSSIONS

ANALYSIS OF ARCH DAMS BY THE TRIAL LOAD METHOD

Discussion*

By Messrs. B. F. Jakobsen, and George R. Rich.

B. F. Jakobsen,† M. Am. Soc. C. E. (by letter).‡—This paper is interesting because it describes an earnest attempt to develop a rational theory of arch dam design. Such a theory must include both the arches and the cantilevers. In order to effect this, the authors assume that the arch dam is a continuous structure, not divided by contraction joints or cracks, that the material is homogeneous and isotropic, and that the foundations are unyielding. To insure freedom from shrinkage cracks, the authors state that openings will be left in the dam and that these will be filled in the coldest weather.§ Otherwise, no provisions are contemplated to eliminate the effect of shrinkage. The writer has dealt extensively with this subject‖ and in his opinion shrinkage cannot be ignored.

When studying the stresses in a structure, it is permissible to exclude from consideration factors of small importance, especially if their effect is to increase the safety of the structure, so that the error committed is on the side of safety. If shrinkage takes place, it is an error on the side of danger to assume that it does not occur. Even if the probability in favor of no shrinkage is much greater than 0.5, it would still be wise, in the writer's opinion, to assume that it does occur and determine the stresses on that basis. A somewhat higher stress might be allowed so as to compensate for the probability that shrinkage may not occur, or may occur but to an extent less than that estimated.

The division of load between the arches and the cantilevers is greatly influenced by shrinkage, temperature variations, swelling due to water-soaking,

* This discussion (of the paper by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., published in January, 1928, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

‡ Received by the Secretary, January 18, 1928.

§ *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 77.

‖ *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 475.

and the yielding of the foundations. The writer has analyzed a number of dams of his own design by making definite assumptions as to shrinkage, etc., and has concluded that the result was so largely influenced by the rather arbitrary assumptions, that it seemed better to treat the dam as a pure arch dam and then allow somewhat higher stresses in order to take account of the unknown assistance given by the cantilevers. In time, when more is known about shrinkage, it can be (and, of course, should be) included in the calculations.

The cracking of the Mulholland Dam* with consequent leaking, convinces the writer that it is not safe to assume that shrinkage does not exist. The shrinkage is no doubt greatest near the surface where it appears first, but there is no reason to assume that it will not penetrate into the dam.

Fig. 5(c)† shows the final proportioning of the water pressure between arch and cantilevers for Elevation 4830 of Dam No. I. It is interesting to note that the load on the arch is practically uniform from Point No. 7 to Point No. VII and is about 90% of the total water load. This is not true in Fig. 8.‡

In all the authors' diagrams, the load distribution at the abutment is zero on the arch and 100% on the cantilever. This is not correct and may be the result of neglecting the shear deformation in the cantilever and including it in the arches or assuming the abutments vertical for each small arch strip investigated.

The division of load between arch and cantilever at the abutment may be obtained in the following manner:§

In Fig. 17(a) the load, p, acts at a point very close to the abutment; it is held in equilibrium by the shear along the arch, $p - c$, and the shear along the cantilever, c. Being very close to the abutment, p cannot influence the load distribution in the remainder of the dam. The two surface elements, $a = \dfrac{1}{\sin \phi}$ and $b = \dfrac{1}{\cos \phi}$, being infinitely close together, must have the same shear stress, or

$$\frac{p - c}{\sin \phi} = \frac{c}{\cos \phi}$$

which gives,

$$\frac{p - c}{c} = \frac{\tan \phi}{1 + \tan \phi} \qquad (51)$$

From Equation (51), the distribution of p can be calculated. The result is given in Table 7.

Equation (51) is not only valid for a load, p, applied very near the abutment, as p in Fig. 17(a), but it gives directly the ratio of the shear forces in two sections, one horizontal and the other vertical, through a point of the foundation. (See Fig. 17(b).) The shear transmitted to Point A by the cantilever, $A\ B$, and that transmitted to the same point by the arch, $A\ C$, that is,

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 246.

† *Loc. cit.*, p. 68.

‡ *Loc. cit.*, p. 72

§ H. Juilliard, "Influence de l'encastrement lateral dans les grands barrages," *Schweizerische Bauzeitung*, December 3, 1921, p. 271.

the total cantilever and arch shear, must be in the ratio prescribed by Equation (51), and independent of the elastic characteristics of the dam.

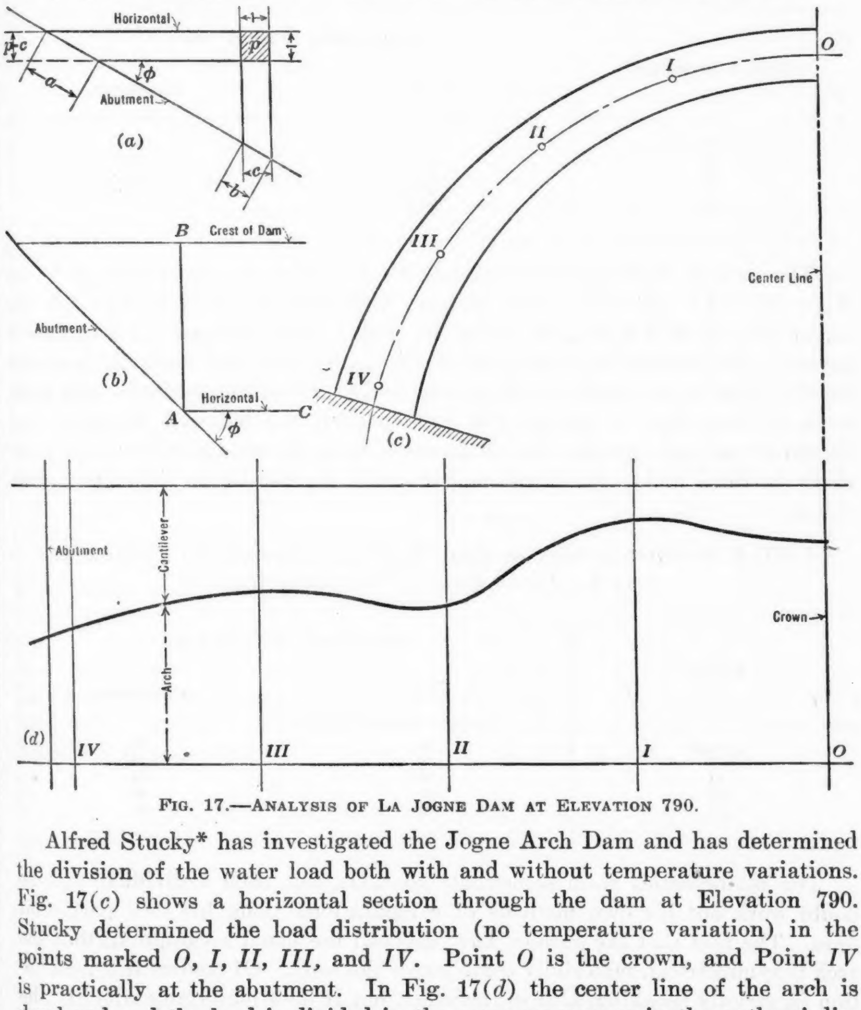

FIG. 17.—ANALYSIS OF LA JOGNE DAM AT ELEVATION 790.

Alfred Stucky* has investigated the Jogne Arch Dam and has determined the division of the water load both with and without temperature variations. Fig. 17(c) shows a horizontal section through the dam at Elevation 790. Stucky determined the load distribution (no temperature variation) in the points marked *O, I, II, III,* and *IV*. Point *O* is the crown, and Point *IV* is practically at the abutment. In Fig. 17(d) the center line of the arch is developed and the load is divided in the same manner as in the authors' diagrams. The values as given by Stucky are shown in Table 8.

Point *IV* at the abutment, having a "50-50" division, corresponds to an angle, ϕ, of 45° (Fig. 17(a)).

At Elevation 770, Point *II* is practically at the abutment and there the arch takes only about 8%; at Elevation 760, Point *I* lies at the abutment and there the arch takes 66% of the water load. The load taken by the arch at the abutment is not constantly zero, as assumed by the authors, but varies between wide limits, depending on the slope at each particular point.

* "Etude sur les barrages arqués," Extrait du *Bulletin de la Suisse Romande,* 1922, F. Rouge & Cie., Lausanne.

TABLE 7.—DISTRIBUTION OF p IN FIG. 17(a), BETWEEN ARCH AND CANTILEVER.

Angle, ϕ, in degrees.	PERCENTAGE OF TOTAL LOAD, p.	
	Arch.	Cantilever.
0	0	100
36	36.6	63.4
45	50.0	50.0
60	63.4	36.6
90	100.0	0

The authors have made an attempt to deal with the secondary arch, see Figs. 10,* 11,† and 15.‡ This leads to difficulties because, in Fig. 15, the angles, $G F R$, $E D S$, etc., are not right angles. The ordinary bending theory assumes the sections to be normal to the center line and leads to incorrect results when it is applied to oblique sections.§ These uncertainties and inaccuracies may have a considerable influence on the bending moments and deformations and involve the whole calculation in serious difficulties, especially in thick arches where the assistance of the cantilever would be appreciable.

TABLE 8.—DISTRIBUTION OF LOAD BETWEEN ARCHES AND CANTILEVERS IN LA JOGNE DAM AT ELEVATION 790.

Points.	PERCENTAGE OF TOTAL LOAD.	
	Arch.	Cantilever.
Crown	80	20
I	86	14
II	56	44
III	60	40
IV	50	50

The mathematics seem somewhat laborious, but each individual will no doubt work out his own methods of computations along his own particular lines. The fact that the authors have dropped the usual assumption, that the arch is symmetrical, materially complicates the work. Of course, this assumption is, strictly speaking, always incorrect, but it does materially simplify the calculations. Judging from Fig. 5, in which the section is not particularly symmetrical and the loading is practically so, the assumption that the arches are loaded symmetrically does not lead one far astray, except perhaps in unusual cases.

The paper is well worth serious study by any engineer interested in arch dams.

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 73.

† *Loc cit.*, p. 75.

‡ *Loc. cit.*, p. 86.

§ "Stresses in Multiple-Arch Dams," by B. F. Jakobsen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 289; see, also, discussion by William Cain, M. Am. Soc. C. E., on p. 315.

GEORGE R. RICH,* ASSOC. M. AM. SOC. C. E. (by letter).†—This excellent paper contributes materially toward a satisfactory evaluation of the many pertinent factors affecting the stresses in an arched dam that has been carefully pressure-grouted near the time of minimum temperature to insure the permanent simultaneous action of both horizontal and vertical elements.

For the short thick arches in the lower sections of the dam, the writer believes that the deflections caused by shearing stresses should not be neglected. He has attempted to show in the following analysis that this effect may be readily included with very little additional labor.

The original notation of the authors has been preserved, with one addition:

Q = the sum of the components of all external forces between the point and the crown, perpendicular to the axis at a given point.

In the formulas given herewith, a subscript, L, refers to the left of the crown and a subscript, R, refers to the right. Table 9, which gives a classification of the various deflection components, is of assistance in writing the basic crown deformations. In this table, E is the modulus for direct stress in all cases. For a rectangular section with parabolic shear distribution, the shear deflection,

$$\Delta = \frac{6}{5} \int \frac{Q\, q\, d\, s}{E_s\, T}$$

in which E_s is the shear modulus. For concrete,

$$E_s = \frac{E}{2.4}, \text{ or } \Delta = 2.88 \int \frac{Q\, q\, d\, s}{E\, T}$$

in which, q is the shear on the differential element, when a load of 1 lb. is applied to the structure at the point where, and in the direction which, the deflection is desired.‡

TABLE 9.—CLASSIFICATION OF DEFLECTION FACTORS.

Bending.	Shear.	Direct stress.
$\Delta \phi = \sum \frac{M\, s}{E\, I}$	$\Delta \phi = 0$	$\Delta \phi = 0$
$\Delta h = \sum \frac{M\, y\, s}{E\, I}$	‡$\Delta h = 2.88 \sum \frac{Q\, s \sin \alpha}{E\, T}$	$\Delta h = \sum \frac{H\, s \cos \alpha}{E\, T}$
$\Delta v = \sum \frac{M\, x\, s}{E\, I}$	‡$\Delta v = 2.88 \sum \frac{Q\, s \cos \alpha}{E\, T}$	$\Delta v = \sum \frac{H\, s \sin \alpha}{E\, T}$

Referring to Fig. 14§ and Table 9, the motion clockwise of the crown section, left half, equals the motion clockwise of the crown section, right half:

* Hydr. Designer, Stone & Webster, Inc., Boston, Mass.

† Received by the Secretary, January 24, 1928.

‡ "Deflections and Statically Indeterminate Stresses," by C. W. Hudson, M. Am. Soc. C. E.

§ *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 79.

$$+\sum_L \frac{M_L\, s}{E\, I} - \sum_L \frac{V_C\, x\, s}{E\, I} - \sum_L \frac{H_C\, y\, s}{E\, I} - \sum_L \frac{M_C\, s}{E\, I} =$$
$$-\sum_R \frac{M_R\, s}{E\, I} + \sum_R \frac{M_C\, s}{E\, I} - \sum_R \frac{V_C\, x\, s}{E\, I} + \sum_R \frac{H_C\, y\, s}{E\, I} \ldots\ldots (52)$$

The downward motion of the crown section, left half, equals the downward motion of the crown section, right half:

$$+\sum_L \frac{M_L\, x\, s}{E\, I} - \sum_L \frac{V_C\, x^2\, s}{E\, I} - \sum_L \frac{H_C\, x\, y\, s}{E\, I} - \sum_L \frac{M_C\, x\, s}{E\, I}$$
$$+\sum_L \frac{H_L\, s \sin\alpha}{E\, T} - \sum_L \frac{V_C\, s \sin^2\alpha}{E\, T} + \sum_L \frac{H_C\, s \sin\alpha \cos\alpha}{E\, T}$$
$$- h_L\, c\, t + 2.88 \sum_L \frac{Q_L\, s \cos\alpha}{E\, T} - 2.88 \sum_L \frac{V_C\, s \cos^2\alpha}{E\, T}$$
$$- 2.88 \sum_L \frac{H_C\, s \sin\alpha \cos\alpha}{E\, T} = + \sum_R \frac{M_R\, x\, s}{E\, I} + \sum_R \frac{V_C\, x^2\, s}{E\, I}$$
$$- \sum_R \frac{H_C\, x\, y\, s}{E\, I} - \sum_R \frac{M_C\, x\, s}{E\, I} + \sum_R \frac{H_R\, s \sin\alpha}{E\, T} + \sum_R \frac{V_C\, s \sin^2\alpha}{E\, T}$$
$$+ \sum_R \frac{H_C\, s \sin\alpha \cos\alpha}{E\, T} - h_R\, c\, t + 2.88 \sum_R \frac{Q_R\, s \cos\alpha}{E\, T}$$
$$+ 2.88 \sum_R \frac{V_C\, s \cos^2\alpha}{E\, T} - 2.88 \sum_R \frac{H_C\, s \sin\alpha \cos\alpha}{E\, T} \ldots\ldots (53)$$

The motion to the right of the crown section, left half, equals the motion to the right of the crown section, right half:

$$+\sum_L \frac{M_L\, y\, s}{E\, I} - \sum_L \frac{V_C\, x\, y\, s}{E\, I} - \sum_L \frac{H_C\, y^2\, s}{E\, I} - \sum_L \frac{M_C\, y\, s}{E\, I}$$
$$- \sum_L \frac{H_L\, s \cos\alpha}{E\, T} + \sum_L \frac{V_C\, s \sin\alpha \cos\alpha}{E\, T} - \sum_L \frac{H_C\, s \cos^2\alpha}{E\, T}$$
$$+ l_L\, c\, t + 2.88 \sum_L \frac{Q_L\, s \sin\alpha}{E\, T} - 2.88 \sum_L \frac{V_C\, s \sin\alpha \cos\alpha}{E\, T}$$
$$- 2.88 \sum_L \frac{H_C\, s \sin^2\alpha}{E\, T} = - \sum_R \frac{M_R\, y\, s}{E\, I} - \sum_R \frac{V_C\, x\, y\, s}{E\, I}$$
$$+ \sum_R \frac{H_C\, y^2\, s}{E\, I} + \sum_R \frac{M_C\, y\, s}{E\, I} + \sum_R \frac{H_R\, s \cos\alpha}{E\, T} + \sum_R \frac{V_C\, s \sin\alpha \cos\alpha}{E\, T}$$
$$+ \sum_R \frac{H_C\, s \cos^2\alpha}{E\, T} - l_R\, c\, t - 2.88 \sum_R \frac{Q_R\, s \sin\alpha}{E\, T}$$
$$- 2.88 \sum_R \frac{V_C\, s \sin\alpha \cos\alpha}{E\, T} + 2.88 \sum_R \frac{H_C\, s \sin^2\alpha}{E\, T} \ldots\ldots (54)$$

The terms involving shear deformations can be easily identified by the coefficient, 2.88. By transposing, the simultaneous equations for the crown forces are obtained. In setting up numerical equations, the terms involving $\sum \frac{s \sin\alpha \cos\alpha}{T}$ and $2.88 \sum \frac{s \sin\alpha \cos\alpha}{T}$, may be combined; in this deriva-

tion they have been left separate to show, by comparison with the authors' original, what terms represent shear deformations:

$$M_C\left(\sum_L \frac{s}{I} + \sum_R \frac{s}{I}\right) + H_C\left(\sum_L \frac{s\,y}{I} + \sum_R \frac{s\,y}{I}\right)$$

$$+ V_C\left(\sum_L \frac{s\,x}{I} - \sum_R \frac{s\,x}{I}\right) = \sum_L \frac{M_L\,s}{I} + \sum_R \frac{M_R\,s}{I} \ldots\ldots (55)$$

$$M_C\left(\sum_L \frac{x\,s}{I} - \sum_R \frac{x\,s}{I}\right) + H_C\left(\sum_L \frac{x\,y\,s}{I} - \sum_R \frac{x\,y\,s}{I}\right.$$

$$- \sum_L \frac{s \sin\alpha \cos\alpha}{T} + \sum_R \frac{s \sin\alpha \cos\alpha}{T} + 2.88 \sum_L \frac{s \sin\alpha \cos\alpha}{T}$$

$$\left. - 2.88 \sum_R \frac{s \sin\alpha \cos\alpha}{T}\right) + V_C\left(\sum_L \frac{x^2\,s}{I} + \sum_R \frac{x^2\,s}{I} + \sum_L \frac{s \sin^2\alpha}{T}\right.$$

$$\left. + \sum_R \frac{s \sin^2\alpha}{T} + 2.88 \sum_L \frac{s \cos^2\alpha}{T} + 2.88 \sum_R \frac{s \cos^2\alpha}{T}\right)$$

$$= \sum_L \frac{M_L\,x\,s}{I} - \sum_R \frac{M_R\,x\,s}{I} + \sum_L \frac{H_L\,s \sin\alpha}{T} - \sum_R \frac{H_R\,s \sin\alpha}{T}$$

$$+ 2.88 \sum_L \frac{Q_L\,s \cos\alpha}{T} - 2.88 \sum_R \frac{Q_R\,s \cos\alpha}{T} - c\,t\,E\,(h_L - h_R) \ldots (56)$$

$$M_C\left(\sum_L \frac{y\,s}{I} + \sum_R \frac{y\,s}{I}\right) + H_C\left(\sum_L \frac{y^2\,s}{I} + \sum_R \frac{y^2\,s}{I}\right.$$

$$+ \sum_L \frac{s \cos^2\alpha}{T} + \sum_R \frac{s \cos^2\alpha}{T} + 2.88 \sum_L \frac{s \sin^2\alpha}{T}$$

$$\left. + 2.88 \sum_R \frac{s \sin^2\alpha}{T}\right) + V_C\left(\sum_L \frac{x\,y\,s}{I} - \sum_R \frac{x\,y\,s}{I}\right.$$

$$- \sum_L \frac{s \sin\alpha \cos\alpha}{T} + \sum_R \frac{s \sin\alpha \cos\alpha}{T} + 2.88 \sum_L \frac{s \sin\alpha \cos\alpha}{T}$$

$$\left. - 2.88 \sum_R \frac{s \sin\alpha \cos\alpha}{T}\right) = \sum_L \frac{M_L\,y\,s}{I} + \sum_R \frac{M_R\,y\,s}{I}$$

$$- \sum_L \frac{H_L\,s \cos\alpha}{T} - \sum_R \frac{H_R\,s \cos\alpha}{T} + 2.88 \sum_L \frac{Q_L\,s \sin\alpha}{T}$$

$$+ 2.88 \sum_R \frac{Q_R\,s \sin\alpha}{T} + c\,t\,E\,(l_L + l_R) \ldots\ldots\ldots\ldots (57)$$

The authors' Equation (29)* gives the temperature term as $\frac{c\,t\,E}{s}\,(l_L + l_R)$. The writer believes that this should be $c\,t\,E\,(l_L + l_R)$ as given in Equation (57). It can be readily seen that these equations are identical with those of the authors, with the exception of the terms added for shear movements.

After working through the various differential deflections, the writer has some appreciation of the difficulty of avoiding errors in algebraic sign in spite of careful checking. For this reason he would ask that Mr. Howell verify these additions to Equations (27),† (28),* and (29), and then compute the division of load for a thick, short-radius arch (such as the Gibson Dam at

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 84.
† *Loc. cit.*, p. 83.

Elevation 4530), first, considering shear deflections, and second, neglecting shear deflections so that a comparison may be available to the profession.

In computing the movements of thick cantilever elements, the same attention should be given to each of the deflection groups: For bending, $\int \frac{M\,m\,d\,s}{E\,I}$; and for shear, $\frac{6}{5} \int \frac{Q\,q\,d\,s}{E_s\,A}$; in which, m and q are the moment and shear, respectively, on the differential element caused by a load of 1 lb. acting on the structure at the point where, and in the direction which, the deflection is desired. M and Q are the general expressions for moment and shear, respectively, on any differential element due to the external loads. The writer would feel indebted to Mr. Howell for incorporating this deformation in the numerical example requested.

The writer hopes that the final closure of the dam by pressure-grouting will eventually disclose great latent possibilities for fixing the limiting stresses in the arches under the assumed range of temperature. This case appears somewhat analogous to the erection of a spandrel-braced steel arch bridge. After starting erection by cantilevering from back-stays, the bottom chord of the bridge is closed at the crown as a three-hinged arch for dead load; then hydraulic jacks are inserted into the top chord; stresses are induced corresponding to the two-hinged condition for dead and live load, and the final riveting is done under the two-hinged condition. A prohibitive length of time is required for the total dissipation of the chemical heat liberated by the setting of large masses of concrete in an arch dam; in addition, a fast construction schedule may make a closure in the summer months imperative. Under such conditions, it would still be at least theoretically possible to preclude vertical shrinkage cracks by inducing compressive stresses of magnitude sufficient to cancel the tensions caused later by shrinkage. The effect of a rise in temperature in raising compressive stresses would also have to be carefully anticipated; but the compressive stresses seldom reach their limiting values, and a given increase in unit compressive stress is much more readily accommodated than the occurrence of a tension of equal magnitude.

The adequate drainage both of the dam proper and the foundation rock, and also the insertion of grout pipes in the bed-rock near the heel, prime requirements of every modern design, amply warrant the uplift assumption used by the authors for the cantilever elements, namely, full head-water pressure at the heel diminishing to zero at the toe, and acting over one-half the base section.

The present indications are that foundation deformations do have an appreciable effect on the division of load between arches and cantilevers. Similarly, when the phenomena of water-soakage are more clearly defined, commensurate refinements can be applied to this valuable method of attack.

The writer is thoroughly sympathetic with the authors' conclusions.* It is to be hoped that ultimately a rational theory embracing all the really pertinent elements can be made available in the form of curves; in the pursuit of this ultimate object, the paper deserves careful study.

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 78.

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PAPERS AND DISCUSSIONS
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GENERAL CONTRACT SYSTEM VERSUS SEGREGATED CONTRACTS

Discussion*

By Messrs. Edward W. Bush, William T. Lyle, and B. F. Jakobsen.

Edward W. Bush,† M. Am. Soc. C. E. (by letter).‡—The author very admirably presents a subject which can well be studied at this time by engineers, architects, and others in charge of the letting of contracts, because, during recent years, there have been organized campaigns in certain localities to require contracts for public buildings to be let by the segregated method. Many reputable sub-contractors doing plumbing, heating, roofing, electrical work, etc., have claimed that they frequently do not get fair prices for their parts of the work because, after the general contractor is awarded the job, he makes the sub-contractors come down to the figures presented to him by, perhaps, a group of irresponsibles. Such practice is deplorable and against the best interests of the construction industry considered from the point of view of the owner, the engineer, or architect, or the contractors interested. On the other hand, trying to cure this evil by letting segregated contracts is substituting a worse practice for one not as bad.

Other methods can and should be used to correct these bad practices. Even where a general contractor does pinch the sub-contractors a little, the owner still has the general contractor's contractual obligations standing between him and any loss or damage caused by a sub-contractor not doing his part. The right kind of a general contractor will select the right kind of sub-contractors and very often they have worked together for years and each knows just how he fits into the other's organization. This makes for economy, which is reflected in the price paid by the owner as well as the profits obtained

* This discussion (of the paper by Ward P. Christie, Assoc. M. Am. Soc. C. E., published in February, 1928, *Proceedings* and presented at the meeting of March 7, 1928), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ Received by the Secretary, February 8, 1928.

by each contractor. No engineer or architect can fit a sub-contractor into a general contractor's organization as well as the latter, and the work will suffer, as explained so well by the author, unless there is a centralized authority and responsibility. The owner has the best chance to obtain the maximum value for the money expended if he gives a building contract to one bidder after competitive bidding. This is the time-tested method under which so much work has been and will be performed.

It is known by the writer that most of the high grade general contractors doing building work possess the requisite knowledge to estimate closely the cost of the portions of the work which will be done by sub-contractors and such cost estimates are actually prepared as a check on the sub-contractor's bids. Unless the engineers and architects have an estimating ability equal to that of the general contractors, the owner's interests are not conserved by segregated contracts.

Referring to the 5% mentioned by the author* as being added by the general contractor to the bid of the sub-contractor, it is of interest to compare this to the 5% fee very generally allowed receivers and administrators of estates on all monies coming in and going out, and these receivers and administrators are also allowed expenses and the employment of expert services if such are needed. It requires a high degree of technical ability and much tact to keep the sub-contractors of a large building job up to the mark on progress and quality of work, and if the services rendered by the general contractor were compensated for in proportion to their true value, as compared with the value often allowed by law for the services rendered by receivers or administrators, the contractor could add, not 5% to the sub-contractor's bids, but 20 to 50 per cent. The small fee actually charged by the general contractor is gross and from it must come the overhead expenses, so his fee really nets him a very small amount of profit, if any. For this fee the general contractor must assume full responsibility on the work performed by the sub-contractors. The owner must necessarily pay some one to exercise the authority needed to correlate the efforts of several organizations, and it is to his advantage to place this authority squarely on the general contractor rather than attempt by segregated contracts to save a little and make the engineer or architect carry the authority wihout indemnification to the owner should anything go wrong.

Even if the general contractor does add 5% to the bids of sub-contractors when making up his bid, he may not add very much more than this to the cost estimate of the part of the work to be done with his own payroll. It is the net profit he will take from the entire job which interests the owner rather than what precise percentage is added to the estimated cost of the various parts. The author has explained that building construction work is generally taken on a small margin of net profit. The writer, after reviewing a large number of contractor's financial statements, confirms this opinion.

A few years ago the contractors bidding on a large school were instructed to file separate bids for all or some of the parts, such as the main structure,

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 402.

the plumbing, the heating, the plastering, etc. They were told that the entire work would be let to one general contractor at an amount determined by adding the lowest bids received on the component parts; and if the general contractor did not choose to award sub-contracts to those who put in the lowest figures, these figures, nevertheless, governed the price the general contractor would receive for these parts of the work. It is difficult to imagine a more vicious form of a letting as it would place full responsibility on the general contractor and, at the same time, take from him the right to select the sub-contractors whose work he guaranteed.

WILLIAM T. LYLE,* M. AM. SOC. C. E. (by letter).†—The writer believes it to be unwise for an engineer to take the attitude of proponent or opponent of either contract system. Each has its advantages and each, its disadvantages. The author, who favors general contracts, states‡ that "some engineering projects have natural segregations of work entirely independent of each other, with respect to interlocking, which makes them suitable for separate contracts". With this unquestionable statement, he dismisses segregated contracts from his analysis and proceeds to establish the merits of the general system.

The author establishes a good case for general contracts in building construction, basing his argument on the interlocking character of the work; but the question will arise as to when and to what extent the elements of a construction job are interlocking. In analyzing this question three considerations arise: First, the parts of the work may be so far removed in space and relationship as to be suitable for segregated contracts, as the author thinks proper; second, they may be so closely related as to constitute an organic whole for which the general contract only is suitable; and, third, they may lie between these extremes, either toward the former or toward the latter. The engineer in charge must make the decision.

It is natural for an engineer to favor the general type of contract. The writer has usually preferred it for the simple reason that responsibility for the timing of the work, legal responsibility, accident prevention, etc., is removed to a considerable degree from his shoulders and placed on those of the general contractor. In company with many of his fellow practitioners, he may have sometimes erred in this preference. The personal convenience of the engineer may not be for the best interests of the owner.

In discussing this question the professional and business equipment of the engineer must be considered. There was a time when, as a mere technician, he was poorly equipped to act as executive, but that time is passing. The modern engineer is taking to himself functions of the promoter and executive and thereby fitting himself for many kinds of important work for which he was formerly poorly qualified. He is becoming more and more a man of affairs. This change in the make-up of the practical engineer is reflected in the professional schools wherever increasing emphasis is placed on the preparation of a humanistic, economic, and administrative character.

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† Received by the Secretary, February 14, 1928.

‡ *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 397.

The writer does not favor the categorical "no" of this paper. There are many large jobs, and smaller ones too, interlocking to a certain degree, and yet so spaced or spread out as to be adaptable to the segregated contract. The engineer in charge of such contracts will have more responsibility but, at the same time, can render greater service. By letting separate contracts, small contractors will have a chance to deal with the owner direct and a double profit may be eliminated. At any rate the work performed will probably be of a better quality. It is often true that the small contractor prefers to deal with the owner direct rather than with a general contractor of whose business character he is suspicious.

The writer is not undertaking to state any "yes" or "no" in the matter, but merely to emphasize the importance of weighing the pros and cons, in cost, security, and promptness of execution. He believes that both systems are useful.

B. F. JAKOBSEN,* M. AM. SOC. C. E. (by letter).†—The author somewhat forcibly emphasizes the advantages of the general contract system, as a system. The writer's experience leads him to believe that the important element is the ability and experience of the particular general contractor as against the particular construction engineer. The general contractor assumes certain risks and responsibilities, but the owner pays for these in any case and it is part of the construction engineer's duties to see that these things are properly arranged.

The author admits that in some cases engineers have performed management functions successfully, not by reason of their technical skill and training, but in spite of it.‡ It is difficult to understand this statement and it is not clear just what the author intends to include in his expression, "technical skill and training". Of two men, one of whom has technical training and the other has not, but who are otherwise equal, there can be no doubt about which one is the more desirable. Some of the most successful contractors the writer knows are technical graduates, and this training does not seem to have affected adversely their ability to understand the management business.

In fact, it would seem self-evident that the executive who understands the reasons for the design, would be *prima facie* better qualified to carry it out. It is quite true that it is pitiful to see a theoretically competent man dabble with practical problems with which he has no experience; but it is still more pitiful to see the practical man dabble with self-conceived theories and insist that designs must be changed to accord with these strange notions of his. The latter case is of much greater frequency than the former, and when the purely practical man has gotten some theoretical conception in his head, based upon his many years of experience, he is generally not amenable to reason. On the other hand the manager of construction who has a fair understanding of the theories involved in the design, as well as of the factors influ-

* Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

† Received by the Secretary, February 23, 1928.

‡ *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 398.

encing construction, is often in a position to assist the designer materially and thus affect a saving without other adverse effects.

The danger of over-emphasizing the advantage of the general contractor is that undue weight is likely to be given to the financial details at the expense of realities. Good accounting is valuable, but the accountant must always understand that his work is after all of a secondary nature. Many, if not all, of the larger engineering corporations, which claim to be management experts are intrinsically financial experts rather than construction managers. Judging from such work as the writer has seen, neither the quality nor the cost of their work has been such as would lead any one to give their system any decided preference.

When the Constitution of the United States was being formulated, Madison insisted that this was to be a Government of laws, not of men. That may be true in a narrow sense and by comparison, but after all it is men who decide what the laws mean in each particular case and that is what counts; or as Pope said,

> "For forms of Government let fools contest,
> Whate'er is best administer'd is best."

It is quite the same in the construction field; at best a system can make up for very little compared to the ability and skill of the individual in charge. The owner needs to look for the best man rather than for the best system.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE O'SHAUGHNESSY DAM AND RESERVOIR

Discussion*

BY MESSRS. J. ERNEST CARMAN AND B. F. JAKOBSEN.

J. ERNEST CARMAN,† ESQ. (by letter).‡—The excavation incident to the construction of the O'Shaughnessy Dam exposed a section of rocks 90 ft. thick in the Valley of the Scioto River and offered a good opportunity for a detailed study of the formations encountered.

The bed-rock exposed along the river valley at the site of the dam is the Columbus limestone of middle Devonian age. The excavation for the dam, however, exposed the entire thickness of the Columbus, a few feet of the Delaware limestone above, and about 2 ft. of the Monroe dolomite below, the latter belonging to the Lower Monroe, or Bass Island formation, of Silurian age. The rocks encountered were correlated by zones with the standard section of the Columbus limestone as determined by Dr. Clinton R. Stauffer.§

The section as seen in the excavation is as follows:

	Thickness, in feet.
Delaware Limestone:	
Bluish limestone in layers of 3 to 6 in.	3+
Columbus Limestone:	
Zone H. Bluish gray, fossiliferous limestone in layers of 1 ft. to 2 ft. The base is at a very smooth plane	7
Zone G. Bluish gray, fossiliferous limestone in layers of 3 to 5 ft.	19
Zone F. Bluish gray, very fossiliferous limestone with *Spirifer gregarius* (*Spirifer gregarius* Zone)	4
Zone E. Bluish gray limestone that weathers buff. In layers of 12 to 18 in. Upper half fossiliferous	14
Zone D. Grayish brown limestone with bands of chert nodules and some chert layers (Chert Zone). Contains some gastropods	12½

* This discussion (of the paper by John H. Gregory, C. B. Hoover, and C. B. Cornell, Members, Am. Soc. C. E., presented at the Fall Meeting, Columbus, Ohio, October 12, 1927, and published in February, 1928, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Prof. of Geology, Ohio State Univ., Columbus, Ohio.

‡ Received by the Secretary, November 2, 1927.

§ *Bulletin No. 10*, Geological Survey of Ohio, 1909.

Zone C. Missing.

Zone B. Brown dolomitic limestone with some banding and in thick ledges of 3 to 5 ft. The basal 8 ft. is more compact and contains some sand grains........ 30

Zone A. Arenaceous limestone matrix with pebbles of Monroe dolomite and chert (Conglomerate Zone).... ½

Monroe Dolomite:

Ashen gray to drab, laminated, argillaceous dolomite in beds of 2 to 8 in. In part, brecciated, with minute fragments .... 2+

Zone A represents the earliest deposit of the Columbus time as the sea transgressed the eroded, pebble-strewn surface of the Monroe dolomite. The sand grains were either on the surface or were washed in by the encroaching sea. Their characteristics are such as to show that they were derived from the Sylvania sandstone of Northern Ohio. The sand content of this layer at this location is greater than at any other known exposure in Central Ohio.

Zones G, E, and B were broken by vertical and inclined joints, a condition having special significance from an engineering viewpoint, for the foundation of a dam.

Zone H forms the upper few feet of the rock wall on either side of the valley, but the top of the zone is not exposed. At a point 50 yd. east of the east end and at a slightly less distance west of the west end of the concrete portion of the dam the "bone bed" which marks the top of the Columbus limestone was encountered in the excavation for the core-walls of the earth embankment sections of the dam. Beyond these points the Delaware limestone was encountered in the core-wall trenches.

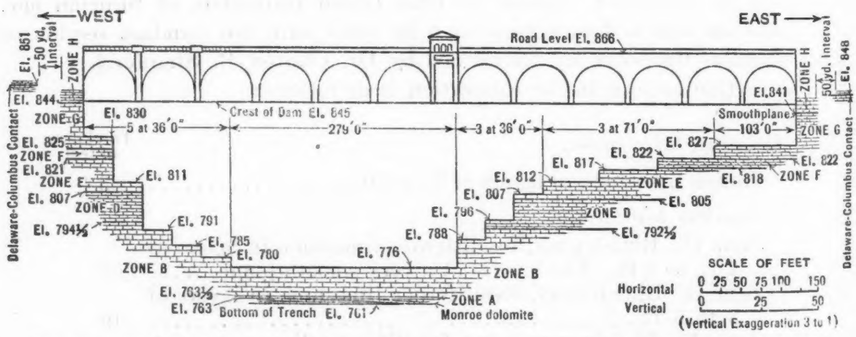

FIG. 25.—CROSS-SECTION OF THE SCIOTO VALLEY AT THE DOWN-STREAM FACE OF THE O'SHAUGHNESSY DAM.

It is of interest to note that the study developed the facts that the Delaware limestone on the east and west sides of the valley is ½ mile apart and the Delaware-Columbus contact is at about Elevation 850 above sea level; whereas, the geological maps, based on the best information previously available, show a separation between the two areas of Delaware limestone of 1½ miles and a Delaware-Columbus contact about 45 ft. higher than was actually found.

Fig. 25, shows a cross-section of the Scioto Valley at the dam, as one sees it looking up stream (north) from just below the dam. The section shows,

to scale, the rock benches just south of the dam and the location of the exposures of the several zones of the Columbus limestone with respect to these benches and to the valley sides.

B. F. JAKOBSEN,* M. AM. SOC. C. E. (by letter).†—The paper indicates that the work was carefully planned and executed, as of course it ought to be. The writer notes the absence of any reference to the water-cement ratio in connection with the concrete. It would be interesting to know how the concrete was tested and what strength was found.

Before concreting was started on the Pacoima and Santa Anita Dams of the Los Angeles County Flood Control District, tests on different concretes were made at the Pacoima Testing Laboratory under the writer's direction. At first, hand-mixing was used, but this was found to give such widely varying results, that no conclusions could be drawn. Therefore, a small motor-driven mixer was installed, and the results were found to check very closely and consistently with the Abrams curve for strength against water-cement ratio. The results obtained at Pacoima have already been reported.‡ All the tests showed that the water-cement ratio was by far the most important feature of concrete production and that when this was watched good concrete resulted.

The desirability of strong and durable concrete has been emphasized often enough and there seems also to be a more or less direct relationship between the impermeability of concrete and its strength. In dam construction the writer thinks that shrinkage may be the most important factor, and it seems well established that the leaner the concrete the less will be the shrinkage. Therefore, for equal strength, the leaner concrete is to be preferred, especially in great structures where shrinkage effects may produce high tensile stresses.

Impermeability is also of great importance, especially in cold countries. A Norwegian engineer, Dr. F. Vogt, has told the writer, that his experience in Europe indicates that in warm climates porous concrete tightens up with age, whereas in cold climates the reverse is the case. This seems to agree with American experience.§

The main part of the O'Shaughnessy Dam is constructed with Class A concrete (see Fig. 14‖), but a 12-in. layer of a richer mix (Class C concrete¶) was added all around the dam. The writer doubts the advisability of adding such a surface coating and is inclined to believe it is detrimental rather than beneficial. This richer mix will shrink more and will probably crack in a great many places, thus tending to produce cracks in the underlying concrete. The writer realizes the desirability of increasing the resistance of the concrete to scour, where it is subject to the action of rapidly moving water, but this ought to be accomplished without increasing the tendency to shrink in setting.

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† Received by the Secretary, February 23, 1928.

‡ *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 583.

§ "Corrosion of Concrete," by John R. Baylis, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 791; also, pp. 792 and 816.

‖ *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 430.

¶ *Loc. cit.*, Table 1, p. 453.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE TREATMENT OF THE WATER SUPPLY OF THE CITY OF COLUMBUS, OHIO

Discussion*

By Messrs. F. H. Waring, George W. Fuller, Robert Spurr Weston, and C. W. Foulk.

F. H. Waring,† M. Am. Soc. C. E.—An important part of the routine of operation at the Columbus water purification plant should be stressed; namely, the practice of continual research on water-softening conducted both in the laboratory and on a plant operation basis. Some items have been incorporated into large scale practice in the more recently constructed water-softening plants. Also, some have been used on a large scale at the Columbus plant. Mr. Hoover deserves great credit for the policy of making available to others the results of his experimentation and researches so that they may reap the benefit of his experiences.

The Use of Carbon Dioxide in Water-Softening Practice.—In 1922 it was demonstrated in the laboratory of the Columbus water purification plant that the addition of carbon dioxide to lime-softened water was beneficial. A considerable portion of the calcium and magnesium carbonates was noticeably difficult to precipitate, as a result of which the sand grains of the filters and the water piping became coated. Some of the incrustation tended to pass beyond the filters and cause trouble in meters and hot-water systems. The introduction of carbon dioxide in the settled water passing to the filters destroys the semi-colloidal portion of the calcium carbonate precipitate by dissolving it as calcium bi-carbonate.

In municipal practice in the Middle West full utilization of this feature was not effected until the Defiance, Ohio, plant, designed by Nicholas S. Hill, Jr., M. Am. Soc. C. E., was placed in service in 1921.‡ Only a brief experience

* This discussion (of the paper by Charles P. Hoover, Esq., presented at the Fall Meeting, Columbus, Ohio, October 12, 1927, and published in February, 1928, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chf. Engr., Ohio State Dept. of Health, Columbus, Ohio.

‡ "The New Water Pumping and Purification Works at Defiance, Ohio," *Journal*, Am. Water Works Assoc., Vol. 7, No. 4, July, 1920.

at the Defiance plant was required to demonstrate the efficacy of this feature of water-softening practice. From the start, the Defiance installation was a tremendous success, accomplishing the protection against incrustation of filters, piping, etc., already mentioned.

In Ohio, the custom has been established of an exchange of experiences among operators in charge of water treatment works by means of annual conferences sponsored by the State Department of Health. Thus, the news of the success at Defiance was rapidly spread within the State; and what was perhaps looked upon at first as experimental, namely, the carbon dioxide treatment, was soon viewed as an essential, accepted process in water-softening practice. Based on the early experiments at Columbus and the full scale practical demonstration on the part of the Defiance plant, Mr. Hoover incorporated the carbon dioxide treatment process in his designs for the municipal water-softening plant at Newark, Ohio. Following the Newark installation in 1924 there have been several carbon dioxide installations in Ohio, each based on previous experience, and, except for minor improvements, largely patterned after the Defiance layout. Delaware completed its plant in 1925; Piqua, Girard, and Greenville, in 1926; and Fostoria, Fremont, and Marion constructed new plants in 1927.

It is pertinent to emphasize that, in addition to preventing incrustation, the carbon dioxide treatment deprives the softened water of a taste given to it by the lime, described by some as a "limey" or caustic taste and by others as a peculiar flat taste. It is practically impossible to tell by taste the difference between any natural water of equivalent hardness and a softened water to which has been applied the proper amount of carbon dioxide gas. From a popular viewpoint, therefore, definite objection to the softening process as applied to municipal practice has been removed.

Bacterial Significance of Softening Treatment.—The promoters of the Columbus plant were pioneers in the field of water-softening. As stated by Mr. Hoover,* very little opportunity was available for those in charge of testing station experiments to study fully the several phases of municipal water softening prior to the construction of the works.

A discovery of very great significance was made after the plant had been put into service. It was found that the water leaving the settling basins after softening treatment and prior to filtration was sterile whenever proper amounts of chemicals required for softening were applied. On further inquiry into the nature of this circumstance it was established that bacterial life was entirely destroyed and prevented if the application of the chemicals resulted in a final product that carried caustic alkalinity. Such alkalinity exists only when free hydrate, termed by some as an excess of lime, is present. It is of interest to note that a similar discovery was made almost simultaneously by Sir Alexander Houston, of London, England, who first showed the possible significance of the excess lime method of treatment from the bacterial standpoint.

As a result of the discovery the Columbus plant has been operating steadily from a water-softening standpoint and not with very much regard for the

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 472.

variations in bacterial content of the raw water being treated. From the bacterial standpoint it remained only necessary to be assured that sufficient lime was added so that the treated water was made slightly caustic.

This experience in the removal of bacteria at Columbus was capitalized by the City of Youngstown in 1918. Mr. Hoover was employed to adjust the treatment process at the Youngstown water purification works so that caustic alkalinity would be obtained in the treated water. The Mahoning River at Youngstown, utilized as the source of raw water supply, carries a tremendous burden of bacteria. For example, the yearly average for 1926, measured in terms of *B. coli* per 100 cu. cm., was 9 100; and the maximum monthly average, likewise expressed, was 23 000. Following the introduction of the excess-lime treatment the final effluent of the plant has been acceptable from a bacterial standpoint without the assistance of chlorination or any other disinfection process. For example, the filtered water average for 1926, in terms of *B. coli* per 100 cu. cm., was 0.2; and the maximum monthly average was 0.6. Other water purification works in Ohio have utilized the excess-lime method for bacterial purposes solely, namely, East Liverpool, Ironton, Portsmouth, all on the Ohio River and carrying even heavier bacterial loads than those given for Youngstown. A prime reason for adopting the excess-lime method for bacterial effects at these cities was the necessity of eliminating and preventing chloro-phenol tastes which occurred occasionally, usually together with the high bacterial loads. The excess-lime treatment made it possible to avoid using chlorine during the intervals that phenol was being experienced in the raw waters at these Ohio River plants.

If raw waters carrying heavy bacterial burdens must be utilized as a source of raw water supply, designing engineers may well give serious thought to the use of the excess-lime treatment to accomplish bacterial reduction, even if no water-softening benefits are desired.

Value of Recently Demonstrated Over-Treatment with Lime from a Softening Standpoint.—One of the features of the Columbus water softening plant is the over-treatment of a part of the raw-water supply; that is, the raw water is divided into approximately equal parts; one receives all the softening reagents; after thorough mixing this over-treated part is mingled with the part of the raw water that had received no treatment, and the whole is given efficient mixing before entering the settling basins.

The reason for over-treating a part of the supply is commonly known to the chemist by the term, "mass reaction". What really happens is that the extra heavy application of lime in the 50% part of the raw water causes the magnesium to come out of solution as magnesium hydrate, a fluffy flocculent precipitate. The magnesium will not come out of the water in such proportion by the action of the lime when only a slight caustic alkalinity is created by the latter; it takes an excess, or over-treatment of the lime to bring about the precipitation of the magnesium. This precipitation is practically complete for that part of the raw water thus treated in the Columbus plant.

Of course, with the processes originally provided it was early understood that such an excess of lime for all the raw water could not be tolerated if the final water was to be palatable and not have a harsh effect on the users

of the supply. Therefore, the part that was over-treated, was neutralized, so to speak, with the remainder of the water receiving no treatment.

The introduction of the carbon dioxide process into water-softening practice has now made it practicable to conduct the mass reaction treatment, or over-treatment with lime, on a 100% scale as applied to the raw water. This is possible because the over-treatment in the lime application is easily compensated by carbon dioxide application and with the resultant precipitation of the excess hydrate alkalinity as crystalline calcium carbonate. In this procedure of water softening, the carbon dioxide is not added in an excess, which would tend to re-dissolve some of the calcium carbonate precipitate, but falls just short of such an amount.

As a result of this system of extra heavy lime treatment, followed by recarbonation, it is possible to precipitate all the magnesium and to throw down the calcium carbonate in crystalline form. The water then coming to the filters carries practically no material to incrust the sand or piping.

Utilization of this feature on a plant scale of operation has been effected regularly since June, 1927, at the Piqua, Ohio, water-softening works. Reports from James Montgomery, Superintendent of the Piqua plant (formerly of the Columbus and Newark Water Departments), have demonstrated the practicability of this process for bringing about further softening of a water supply, coupled with a more pleasing final product than was heretofore possible. Following closely on the success of the Piqua plant, the treatment has been installed regularly at the Greenville, Ohio, plant, under the efficient management of Mr. John Q. McGuire (also formerly of the Columbus Water Department). The total hardness of the Miami River water before treatment at the Piqua plant since June, 1927, has averaged 215 parts per million. The total hardness of the treated water since June has averaged about 50 parts per million. Mr. Montgomery estimates that the treated water would have averaged about 80 parts per million if the works had been operated on the usual plan that was in effect before June. The raw water supply at Greenville is taken from wells having an average total hardness of about 450 parts per million, of which approximately one-fourth represents incrustant hardness. The treated water since July, 1927, has averaged about 90 parts per million, whereas before July the treated water had a total hardness of about 120 parts per million.

It is of interest to note that the new softening procedure, using extra heavy lime treatment, made possible by carbon dioxide application, costs no more at the Greenville and Piqua plants than the old lime-soda method; but with the advantage, however, that the treated water is much softer.

The large scale experiences of Piqua and Greenville have demonstrated the additional value of the carbon dioxide process. As the result of the early experiences at Defiance and Newark, followed by the more recent experiences at Piqua and Greenville, a complete carbon dioxide treatment process has now been installed at Columbus, and probably before very long the new schedule of softening will be adopted at the Columbus works just as it has been at Piqua and Greenville.

The policy of freely exchanging with others the experiences of both research and plant operation has paid Columbus well; and at the same time it has extended great benefit to other municipalities. Mr. Hoover deserves commendation for the liberal application of this policy.

GEORGE W. FULLER,* M. AM. SOC. C. E.—The author is entitled to a great deal of credit for the splendid way in which he has presented his operating experiences with the Columbus plant. It was a pioneer plant in municipal water softening as there were only a few sizable plants of that kind when it was designed.

Without intending to discuss this paper from a technical standpoint, it seems appropriate to say a few words in regard to the development of the art of water softening. In the autumn of 1903, when the late Rudolph Hering, M. Am. Soc. C. E., and the speaker were called to Columbus to assist in formulating a program for water supply and sewage disposal, it was thought that the greater difficulty with those two undertakings would relate to sewage treatment; hence, a sewage testing station was operated on a sizable scale for several months. As Mr. Hoover has stated,† the tests that led to the basic design of the water purification plant were made in a somewhat haphazard way so far as anything outside the laboratory itself was concerned. The work was done in carboys and small tanks. At that time there were only three sizable plants in existence, municipally operated; namely, at, McKeesport, Pa., Winnipeg, Man., Canada, and South Hampton, England, although there were quite a number of industrial plants for water softening.

The main purpose of the water-works improvements was to correct a very highly polluted water. Mr. Hoover speaks‡ of 138 deaths per 100 000 population from typhoid fever. It should be made plain that that situation has been changed so that there are now less than 5 deaths per 100 000 population per year, which is, of course, very gratifying from the public health standpoint.

In regard to the water-softening installation itself, it was not the ambition of the designers to produce a completely softened water, but rather to make the Columbus water supply approach in hardness that of the Great Lakes. In fact, the policy was expressed of operating the plant so as to produce a water of hardness equal to that of Lake Michigan.

In those days at Columbus, with a very hard water from galleries, the community was led to use many thousands of cisterns. The people were accustomed to rain water. They used their pressure-water supply to a substantial extent for operating water motors, whereby soft cistern water was pumped into the house for washing purposes.

When the plant went in service it was found that a hardness of 120 parts per million, or about 7 grains per gal., did not suit the citizens. They wanted a softer water. Particularly, on Mondays, there was a constant ringing of the telephones at the City Hall, the Water Department Office, and the plant,

* Cons. Engr. (Fuller & McClintock), New York, N. Y.

† *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 472.

‡ *Loc. cit.*, p. 471.

to have a much softer water. That led to the experiences which Mr. Hoover has related, and which as the data in his paper will show, resulted in a constantly decreasing hardness down to a point of about 85 parts per million.

With that softening came some burdens, due to the separation of calcium and magnesium carbonates in excess of their natural solubility in water, which is, as the speaker recalls, about 35 parts per million in the absence of carbonic acid. These slowly forming crystalline products accumulated in the pipes, on the sand grains of the filters, in the household plumbing, and actually stopped the meters.

Mr. Hoover has energetically tackled these problems, taking a plant that was intended to be flexible, but not as well prepared in its design as it could be made now. He is entitled to great credit for the persistence with which he has kept up his research work. Every hour during this period of twenty years there has been an analysis made of the water at different stages through this plant, and he has thereby gained experience that entitles his opinion to a great deal of consideration. The speaker is glad to note that this pioneer work has been the forerunner of developments of the art which have been so significant and which have been found to be so well worthy of application in many places in Ohio and elsewhere.

A plant such as Mr. Hoover operates is in a certain sense a manufacturing plant, creating a product which the people like in their private homes, but which is also more suitable for manufacturing and for steam-raising purposes. The speaker is aware that a municipal plant does not always find it desirable or practicable to produce water that best serves various industrial purposes. There are two things here to be considered: First, how far is a municipality justified in producing a better water supply than that which serves, reasonably, the citizens and the water users, at the lowest cost; and, second, to what extent may those who need water for special purposes expect support and consideration from the producers of the municipal supply?

What is referred to particularly, of course, is water for boiler feed. Without doubt, industrial plants in need of a very soft water can get it by local treatment of a portion of the public supply. When it comes to the question of what is the best style of operating a municipal water supply, consideration must be given to those things which Mr. Hoover has described; that is, elimination of all the deposits that are annoying to those who control the plant, annoying in the meters and the service piping and household plumbing. On the other hand, the practice of recarbonation is complained of as increasing the corrosion problem when used as the supply to railroad locomotives and for steam raisers in general.

The speaker will not attempt to offer any fixed conclusions as between those two lines of thought. They do exist, however, and he is sure that Mr. Hoover will consider them in the future, as he has in the past, and that there will continue to come from Columbus a vast fund of helpful information.

Attention is called to the fact that there is in progress in the United States, to-day, a very thorough preparation for an exhaustive study of boiler-

feed water. A Committee of the American Water Works Association has been operating under its Standardization Council for several years. That Committee has made an unusual effort to secure co-operation from the users of water for boiler feed. There is now coming into shape a program that has the joint sponsorship of the American Railway Engineering Association, the National Electric Light Association, the American Boiler Manufacturers Association, the American Society of Mechanical Engineers, and the American Water Works Association, and perhaps some others. These co-operating National organizations have a strong Executive Committee that represents all branches and ramifications of this undertaking in respect to boiler-feed water. It is their purpose to agree upon a budget and program that will enable them to investigate adequately all the main problems arising in water service to railroads, and water for the power plants of the country. These investigations will probably cover a period of 5 or 6 years and involve the expenditure of $250 000. Mr. Hoover may be assured that much fruit will come ultimately from these studies and with them, an appreciation for the work which he has been doing at Columbus, and which he will continue to do.

Robert Spurr Weston,* M. Am. Soc. C. E.—There are not many who realize with what insufficient data the designers of this plant had to work. Softening processes are about 100 years old, but for the most part they have been applied to small works in England that were supplied largely from wells. The experiences at plants like those at Canterbury and South Hampton, England, therefore, were not altogether applicable to the Columbus problem. Furthermore, the waters were largely calcareous rather than dolomitic and, consequently, difficulties due to slimy deposits of magnesia, and poor crystallization of the precipitate, were not so vital as those at Columbus in the early days.

The plants in the United States designed for industrial softening expressed nothing that was novel in the way of treatment and nothing that was based upon modern physical chemistry. The Columbus designers supplemented the data with the meager available results of experiments made in bottles on a very small scale. Yet, in spite of all difficulties, John H. Gregory, M. Am. Soc. C. E., and his advisers succeeded in designing a plant so flexible that it is operating to-day, with some changes to be sure, but such changes have not vitiated the efficiency of the original design in any way.

It is fortunate that Mr. Hoover has been able to conduct his research work as well as he has. Another fortuitous circumstance is a rapidly changing river, which made necessary the employment of a man of Mr. Hoover's character and caliber to supervise the operation of the plant. One could see on visiting the plant in its early days, when the chalky incrustations on the sand grains were a serious detriment to its operation, that the management was hard put to get good results. Failure was often threatening; yet success was attained.

The work of Mr. Hoover in connection with the recarbonation of water has been mentioned, and one can only add praise of his genius to the com-

* Cons. Engr. (Weston & Sampson), Boston, Mass.

mendations of others. Mr. Hoover is also to be commended for the introduction of what almost every one engaged in water-works practice calls "Hoover alum", which effected, not only an economy of the first order from the standpoint of the Columbus plant, but also a much lower price for sulfate of alumina for other plants than if the competitive process had not been devised.

The foresight of those in charge of the Columbus plant in providing for excellent management and in keeping records which are accurate and informing, is to be greatly praised, and all who have had to consult these records appreciate the care and the skill with which they have been collected.

The tendency of modern standards of quality is toward a softer water. A hardness of 60 to 80 parts per million, the goal of most lime-soda plants, is failing to give satisfaction to critical consumers. In many cases, this means going beyond the lime and soda processes and using zeolite softeners, which can remove all the hardening constituents. The Ohio Valley Water Company has recently installed a large plant of this type at McKees Rocks, Pa., by which the hardness is being lowered to a degree that is more pleasing to the consumer than 60 or 80 parts per million, which, in Massachusetts by the way, is considered an extremely hard water. In Brookline, Mass., the water is called hard at 40 parts per million. The average hardness of the water in Boston, Mass., is 12 to 18 parts per million. The standards will surely rise and the combination of the lime-soda and zeolite processes for hard waters will come into more general use, especially where non-carbonate hardness is high and where salt, for the regeneration of the zeolite, is cheap. Even Mr. Hoover may soon begin to compare the cost of salt for reducing the sulfate content of the hardness of the water at Columbus by the zeolite process with that of soda by the lime-soda process. Surely all engineers are greatly indebted to Mr. Hoover for his paper, and the work that has preceded it.

C. W. Foulk,* Esq.—As a matter of interest the speaker presents the following data to show the advantage of softened water to the average householder. The Columbus plant removes from 180 to 190 parts per million of hardness from the Scioto River water. This, if left in the water, would destroy about $6 worth of soap per year in the average household. (The household price of soap is taken at 10 cents per lb.) On the basis of 50 000 households in Columbus, $300 000 worth of soap would be wasted each year. This waste is eliminated by the softened water.

Considering the positive gains arising from the use of soft water, attention is called to the following items in the construction and operation of a house, which are needed in a hard-water city, but not in one with a soft-water supply:

Hundred-barrel cistern	$110
Electric water pump	120
Installing pump	20
Extra plumbing	50
Total	$300

* Prof. of Analytical Chemistry, Ohio State Univ., Columbus, Ohio.

This means that it costs $300 less to build a house in Columbus than to build the same house under the same conditions in a hard-water city. If interest and depreciation are calculated, it is seen that there is an annual saving of from $25 to $30, because of the soft-water supply.

One more observation is worth making. Mr. Hoover has calculated that 1 ton of this Columbus water costs the housekeeper 4 cents. What other valuable commodity can be had at that price per ton, delivered?

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PAPERS AND DISCUSSIONS

INLETS ON SANDY COASTS

Discussion*

By G. T. Rude, M. Am. Soc. C. E.

G. T. Rude,† M. Am. Soc. C. E. (by letter).‡—This paper is of particular interest to engineers and officials of States having shore properties that have increased enormously in value during the past few decades as pleasure resorts. The stability of the inlets is directly connected and dependent upon the stability of the beaches along the ocean front, and, too, the value of a shore as a pleasure resort is enhanced by the existence of a near-by inlet and lagoon. Colonel Brown has covered the subject in an exceptionally thorough and comprehensive manner and engineers engaged on coast-protection operations are indebted to him for this paper.

Colonel Brown states§ that:

"In the open sea, the maximum velocity of tidal currents is found at high and at low water, while the slack or the turn of the tide [current] occurs at mean sea level."

This is a rather sweeping statement and some amplification will probably not be out of place. He probably has in mind the immediate vicinity of inlet openings, because in the open sea the tidal current is not of the rectilinear or reversing type with a period of slack water; nor is the current ever slack, its direction changes constantly in a rotary movement. The varying velocities throughout a tidal cycle, when plotted to scale on polar-co-ordinate paper, approximate an elliptical shape. The times at the extremities of the minor axis of the ellipse; that is, the time of minimum velocity of the current, may be taken to represent the times of slack water of the ordinary rectilinear tidal current. This minimum current, however, does not occur at the time

* This discussion (of the paper by Earl I. Brown, M. Am. Soc. C. E., published in February, 1928, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chf., Div. of Tides and Currents, U. S. Coast and Geodetic Survey, Washington, D. C.

‡ Received by the Secretary, February 27, 1928.

§ *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 510.

the height of the local tide is at mean sea level, basing this statement on the results of the only tide observations made on the open sea, so far as the writer is aware, in the same vicinity as current observations. The U. S. Coast and Geodetic Survey has obtained tidal observations on Five Fathom Bank off the Coast of Delaware, and current observations on Five Fathom Bank Light Vessel. From these observations it is found that the minimum tidal current at Five Fathom Bank Light Vessel occurs about one hour after local high and low water.

The author has described ocean currents due to winds as usually feeble and infers that ocean tidal currents are of much greater velocity.* In general, coastal tidal currents, except at the immediate entrance to estuaries, are of small velocity (seldom exceeding 0.3 knot), while coastal wind-driven currents attain a velocity, at times equal to 1½ knots as determined by actual observations. It has been estimated that alongshore currents close in to the beach, where it is difficult to make actual observations, attain a velocity of 6 to 8 knots.† These high-velocity alongshore currents are confined to a very narrow belt, probably not beyond the breaker line.

The result of observations made at several light vessels along the Atlantic Coast that were suitably located as to estuary entrances, or the Gulf Stream, is shown in Table 2.‡

TABLE 2.—VELOCITY OF TIDAL CURRENTS.

Light vessel.	Velocity of tidal currents, in knots.
Portland Light Vessel	0.20
Boston Light Vessel	0.30
Fire Island Light Vessel	0.25
Northeast End Light Vessel	0.30
Fenwick Island Shoal Light Vessel	0.30
Winter Quarter Shoal Light Vessel	0.20
Cape Charles Light Vessel	0.30
Cape Lookout Shoals Light Vessel	0.30
Frying Pan Shoals Light Vessel	0.40

On the other hand, on the Atlantic Coast of the United States, the velocity of the wind-produced current is about 1½% of the velocity of the wind. (See Table 3.)

TABLE 3.—CURRENT VELOCITY DUE TO WIND, NORTH ATLANTIC COAST.*

Wind velocity, in miles per hour	10	20	30	40	50	60
Current velocity, in knots	0.2	0.3	0.4	0.6	0.8	1.0

* Current Tables, Atlantic Coast, U. S. Coast and Geodetic Survey, p. 80.

It will be seen from Table 3 that a wind velocity of only 20 miles will create a current equal to the average tidal current and that a 60-mile wind

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 510.

† "Problems Involved in Coast Erosion," *Military Engineer*, Vol. XVI, No. 90, November-December, 1924, p. 460.

‡ Current Tables, Atlantic Coast, U. S. Coast and Geodetic Survey, pp. 72–75.

will create a current of 1 knot, which is considerably in excess of the average open-sea tidal current.

Colonel Brown has called attention to the fact that the travel of the littoral drift depends on the lay of the coast with respect to the dominant direction of the waves, and, consequently, of the winds.* In this connection the writer had occasion to study shore changes at Cape Hatteras and an attempt was made to correlate these changes with storm-wave action from local storms.† The original records of the U. S. Weather Bureau Meteorological Station at Cape Hatteras for the period, 1875 to 1920, were consulted. From these studies it appeared that impact of storm-wave action is the prime factor in shore-line changes on the open coast and wind-driven currents the secondary factor or transporting agent.

Fig. 9 shows the shore changes over different periods in the vicinity of Cape Hatteras as determined from surveys made at intervals from 1850 to 1921. The straight shore line, exposed to the predominant north and northeast storms, has made a consistent recession during the entire period, while the shore line on the south side of the point, not directly exposed to this storm-wave action, has steadily progressed southward. The upper arrow-rose in Fig. 9 represents graphically the resultants of the storm winds for the 21-year period from 1900 to 1920, and the lower arrow-rose, the average wind data (including storm winds) for the period, 1875 to 1920. The lengths of the arrows represent the relative strengths of the different components, the solid arrows through the arc of exposure and the skeleton arrows through the protected or offshore arc.

It will be noted from the "roses" that the mean (prevailing) wind movement is fairly well balanced in opposite directions even if the predominant northerly storms are included, whereas the storm-wind components are predominantly north and northeast, and the shore movement has been in a direction in accord with the storm-wind components.

In arranging the prevailing wind data (including storm winds) in Table 4, the mean yearly wind movement has been taken from the original meteorological records in groups for each cardinal and inter-cardinal point. The resultants were obtained, and the prevailing wind arrow-rose (Fig. 9) was constructed.

The storm-wind data extend back only to 1920 in the meteorological records, the storms not having been recorded in earlier records. In compiling these storm data for Table 5 on which the storm arrow-rose (Fig. 9) is based, the U. S. Weather Bureau's definition of a storm wind was accepted. This velocity is now taken at 40 miles per hour; that is, this velocity is attained each hour and maintained for at least 5 min., and storm-warning signals are displayed for winds of that velocity. Prior to 1906, it was 33 miles per hour; therefore, the latter figure was taken for the entire period in order to make the data comparable.

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 508.

† *Annals*, Assoc. of Am. Geographers, Vol. XII, pp. 87–95.

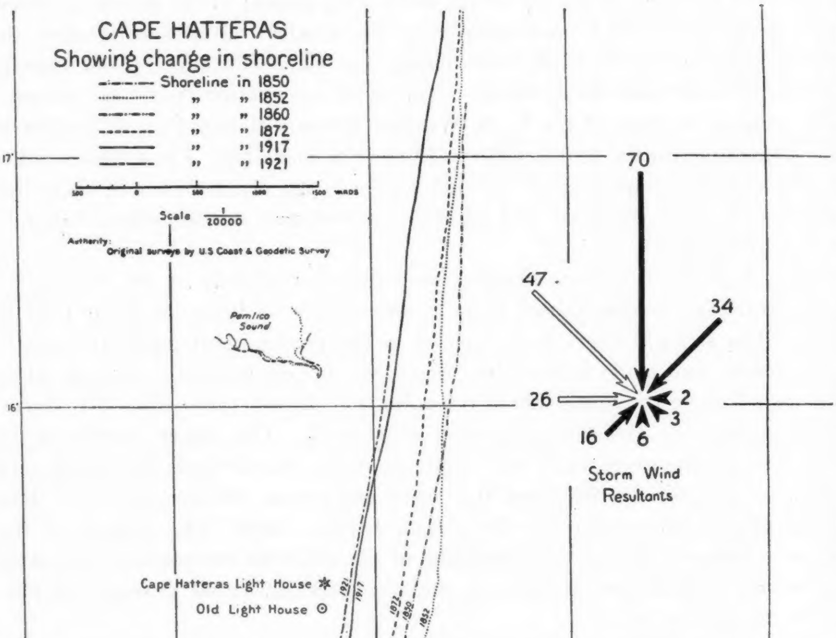

FIG. 9.—CHANGES IN SHORE LINE, CAPE HATTERAS.

TABLE 4.—PREVAILING WIND MOVEMENT, CAPE HATTERAS METEOROLOGICAL STATION.

Period.	Average yearly total wind movement, in mile-hours.	AVERAGE RELATIVE FREQUENCY OF WINDS FROM:							
		North.	North-east.	East.	South-east.	South.	South-west.	West.	North-west.
1875-1884	120 390	95	340	62	95	87	224	76	81
1885-1894	116 295	167	160	52	47	121	155	95	80
1895-1904	122 474	137	153	43	46	79	144	86	66
1905-1914	128 216	94	166	44	43	60	189	69	72
1915-1920	129 380	118	149	52	41	72	167	69	63
Means.........	....	122	194	50	54	84	176	79	72

In summing the storm-wind velocity-hours for each cardinal and inter-cardinal point, only those hours were included which were of a 33-mile velocity or more, and then only for days during which the total wind movement aggregated 600 wind-velocity-mile hours, or more. That is, a squall of just a few hours' duration was neglected, because no sizable sea is likely to be created by such a storm. Since the total wind movement for different days varies from about 100 to about 1 200 velocity-mile hours, 600 was taken as the minimum total wind movement which would best define a storm day. In the case of storm winds, the number of storm hours were summed for each cardinal and inter-cardinal point instead of summing the absolute velocity values for the different hours.

TABLE 5.—STORM WIND RESULTANTS FOR DIFFERENT PERIODS.

Period.	AVERAGE NUMBER OF STORM HOURS WITH WINDS FROM:							
	North.	North-east.	East.	South-east.	South.	South-west.	West.	North-west.
1900 to 1920	70	34	2	3	6	16	26	47
1900 to 1910	59	41	2	3	6	15	17	46
1910 to 1920	78	26	1	3	6	17	33	49

Considering the apparent variability of storms, the close agreement between the resultants of the storm winds for each cardinal and inter-cardinal point for different periods is rather remarkable when taken over comparatively long intervals and tends to indicate that the force exerted on the coast line by storm waves is not the variable factor sometimes ascribed to it.

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PAPERS AND DISCUSSIONS

SILTING OF THE LAKE AT AUSTIN, TEXAS
Discussion*

By Messrs. L. M. Lawson, C. H. Eiffert, and Oren Reed.

L. M. Lawson,† M. Am. Soc. C. E. (by letter).‡—The author has described one of the important features of the storage of water in the Southwest where inflows carry a large percentage of sediment. An opportunity for similar study presented itself in the creation of the Elephant Butte Reservoir on the Rio Grande. The Elephant Butte Dam forms one of the largest reservoirs in the United States for irrigation, and during the writer's superintendency of the Rio Grande Project (of which this dam is the main storage feature), two silt surveys were made and interesting data collected on the amount of sedimentation.

The Rio Grande, which is the source of the water supply for the Rio Grande Federal Irrigation Project, rises in Southern Colorado and has a drainage area of 30 000 sq. miles above San Marcial, N. Mex., which point is the beginning or upper end of the Elephant Butte Reservoir. While the spring run-off from melting snows furnishes the larger part of the inflow into the reservoir, violent fluctuations in discharges are caused by late summer rains. The highest recorded discharge of the river at San Marcial was 33 000 cu. ft. per sec. Usually, about the middle of July, the flow reaches zero.

The reservoir formed by the Elephant Butte Dam has a maximum capacity of 2 638 800 acre-ft.; a length (when full) of 40 miles; a maximum width of 6 miles; and a shore line of about 200 miles. The maximum depth of water is 200 ft. Previous to the construction of the dam, topographic maps were made of the reservoir site. The first storage was accomplished in 1915 and

* This discussion (of the paper by T. U. Taylor, M. Am. Soc. C. E., published in February, 1928, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Commr., U. S. International Boundary Comm., El Paso, Tex.

‡ Received by the Secretary, February 14, 1928.

the dam was completed in 1916. In 1920, taking advantage of the unusual large storage in the reservoir, a silt survey was made on cross-sections previously established. Again, in 1925, these cross-sections were re-sounded and the location and amount of sedimentation were fairly accurately ascertained.

The total river discharge into the reservoir as measured at San Marcial, for the period, November, 1916, to August, 1925, was 10 840 500 acre-ft. The total silt deposited for the same period, as found by silt surveys, was 177 740 acre-ft. The percentage of deposited silt to water is 1.6, and the average annual deposit for the period under observation is 20 470 acre-ft.

While practically all the sediment remains in the reservoir site, a large percentage is deposited above the ordinary usable storage elevation.

C. H. Eiffert,* M. Am. Soc. C. E. (by letter).†—The rate at which the capacity of the Austin Reservoir has been reduced is truly remarkable. In the retarding basins of the Miami Conservancy District the rate of silting is comparatively very small; however, a study of it has recently been begun.

The manner of silting in these basins will differ greatly from that described by Dean Taylor. Water is stored only temporarily during floods. Most of the time they are entirely empty, and practically all the included area is good agricultural land and is farmed every year. To this farm land the deposit of silt is of great value as fertilizer.

The annual rate of deposit will not be uniform because the number of floods and their intensity varies considerably from year to year. The amount of silt carried will undoubtedly vary quite a little according to the season at which a flood occurs and also according to the intensity of the rainfall and the condition of the soil and crop growth at the time of flood. During the intervals between floods there will be a great many local rains, which, while they will not cause back-water in the basin, will result in considerable run-off and local erosion, thus tending to remove much of the silt deposited by back-water.

In two of the basins where the rate of silting is likely to be greater than in the others, a number of cross-sections have been taken. This work will be repeated from time to time and the amount of silt remaining as a permanent deposit will be determined. In addition, a number of silt-catching boxes have been installed. By means of these it is intended to obtain the actual amount of silt deposited by individual floods for comparison with the amount remaining permanently as determined by the cross-sections. The variation in the amount of silt deposited by floods of approximately equal severity at different seasons may also be determined. Samples may also be used for analyses in order to determine the value of the silt as fertilizer.

The boxes that are being used experimentally are 1 ft. square and 6 in. deep. The bottom of each box extends out 1 ft. on one side of the box. The deposit will be measured in the box and on this extension of the base.

To this time, during a period of about seven years, the amount of silting has been inappreciable in its effect on the capacities of the basins.

* Chf. Engr. and Gen. Mgr., The Miami Conservancy Dist., Dayton, Ohio.

† Received by the Secretary, March 2, 1928.

Oren Reed,* Assoc. M. Am. Soc. C. E. (by letter).†—The silting of the Austin Reservoir in a relatively short time is a forceful demonstration of the enormous débris and silt load of the rivers draining the Great Plains. The value of a storage reservoir depends on its permanency. The rate at which erosion of soil is taking place on the basin of the supplying stream is a major factor in determining the period of service of a reservoir.

In many cases, as at Elephant Butte, silting has been accepted as a natural condition and its effect has been fully taken into consideration. The loss in investment through the silting up of a storage reservoir is small when compared with the actual loss to the region of the reservoir site. For this reason, it is of paramount importance that, wherever it is possible, the permanence of a reservoir be assured through adequate protection of the stream basin from erosion.

The Colorado River (of Texas) heads in Eastern New Mexico near the Texas line, at an elevation of less than 4 000 ft. It flows southeastward and crosses the Balcones fault zone a short distance above Austin. The Balcones escarpment is the result of an enormous geologic fault that extends across the entire State from the Red River to the Rio Grande. It marks a sudden change from the gently sloping Rio Grande Plain to the eroded plateaus. The clay soil cover of the water-shed supports a fair growth of scrub timber. The head-waters of the Colorado flow in the deeply fissured limestone of the central plateau and are normally clear. They carry, however, during periods of flood, an enormous quantity of silt eroded from the gorges and channels of the small streams which indent the plateau region. The silt burden has been roughly estimated at 1% of its volume.‡ Wide fluctuations between minimum and maximum flow are caused by periods of intense rainfall.

It is generally accepted that a forest is an excellent agency to prevent soil erosion. The streams in the Western States, that have their sources in the forested mountains, are largely clear, or of prevailing low turbidity, until they enter the treeless plains. The thin soil cover and low rainfall in the Upper Colorado Basin are not favorable to the growth of a forest cover. In many places it is impossible to maintain a sufficient cover of shrubs and grasses to afford even moderate protection against erosion of the soil.

The use of check dams or settling basins on the head-water streams however, would be of great value. Rough alluvial lands which have been injured by gullying during floods, or upon which sand or gravel bars have been deposited, could be flooded by low and inexpensive dams. Most of the débris and silt carried by the stream would be thus deposited by natural elutriation. For several years check dams have been used to control the flood flow of the upper water-sheds of certain torrential streams in Southern California.§ This was an adaptation of the Swiss and Austrian method of torrent control by means of these dams. Channel rectification and levee protection in the Upper

* Asst. Designing Engr., San Joaquin Light & Power Corp., Fresno, Calif.

† Received by the Secretary, March 3, 1928.

‡ *Bulletin No. 1430,* U. S. Dept. of Agriculture.

§ *Engineering News-Record,* May 13, 1916.

Rhine Valley were largely futile until such dams were built to lessen the work of erosion.

All protective cover, whether of grass, brush, or forest, on the head-water streams should be carefully maintained and bank protection should be utilized where economically feasible. For the Great Plains, from Texas northward, about the only practicable means of reducing soil erosion and silting of reservoirs and watercourses is to construct and maintain a comprehensive system of check dams on the head-water streams. The flood flow which at present causes enormous damage to property and life at frequent intervals, would be in part absorbed and retained in ground storage. The increased ground storage would be reflected in a higher minimum flow in the rivers.

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PAPERS AND DISCUSSIONS
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UPWARD PRESSURES UNDER DAMS: EXPERIMENTS BY THE UNITED STATES BUREAU OF RECLAMATION

Discussion*

By E. L. Chandler, M. Am. Soc. C. E.

E. L. Chandler,† M. Am. Soc. C. E. (by letter).‡—The subject covered by this paper is very interesting and the writer wishes to offer some data from his own observations in an effort to add further material to the excellent contribution of Mr. Hinds.

Some enlightening data have been gathered as the result of work done on the Pinhook Dam at Maquoketa, Iowa. This dam and the power house form a hydro-electric development about 30 miles from the mouth of the Maquoketa River, a tributary of the Mississippi. It was built by the Eastern Iowa Power Company in 1923-24, ponding of the water having been started in December, 1923. Fig. 21 is a general view of the dam and reservoir looking up stream toward the west. The structure consists of an earth dike about 20 ft. high, extending from the north bank of the river about 400 ft. northerly to the hills on that side of the valley; a concrete spillway section approximately 160 ft. long; and a power house abutting the south bank. The spillway is a hollow deck, reinforced concrete structure, of which Fig. 22 shows a typical cross-section. It will be seen that the spillway is divided into six bays, each bay being fitted with a Taintor gate. For the purposes of this description, the bays will be numbered 1 to 6, from left to right in Fig. 21. Outcropping limestone is found on each side of the valley, but the river bed consists of sand and gravel to an unknown depth. Therefore, all the concrete structures were founded on timber piles from 20 to 30 ft. long, and along the up-stream line of the spillway and power house a 30-ft., steel, sheet-piling cut-off was driven.

* This discussion (of the paper by Julian Hinds, M. Am. Soc. C. E., published in March, 1928, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† With Price Bros. Co., Dayton, Ohio.

‡ Received by the Secretary, February 20, 1928.

During construction, pipes were installed for the purpose of measuring upward pressures under the spillway. They were placed in systems at right angles to the axis of the dam in Bays 2 to 6. (Fig. 21.) Each set consisted of five pipes, 1½ in. in diameter, lettered from *A* to *E* in Fig. 23. Each pipe terminates at its lower end in a pocket of sand and gravel so graded as to permit easy flow of water. Pipes *A* to *D* extend upward into the chamber under the deck of the spillway, and each pipe is capped with a reducer and a ¼-in. brass air-cock. Whenever it is desired to obtain a reading at any station, a suitable length of rubber hose, fitted over a short ¼-in. nipple at one end, is coupled to the air-cock which is then opened. In order to determine the head acting, it is only necessary to raise the free end of the hose until the elevation is found at which water ceases to escape, and then, by measuring up from the known elevation of the concrete floor, it is a simple matter to reduce the results to a known datum. Readings in Pipe *E* are made by direct measurement in the pipe from the top of the tumble-bay weir.

The amount of percolation through any deposit of sand and gravel depends considerably, of course, on the character of the finer portions of the material. The tabulation that follows, is a mechanical analysis of a sample taken from the site of the Pinhook Spillway before construction was started. Considerable intermixed larger gravel, and occasional boulders were encountered in the course of construction, but the analysis is believed to be representative of that part of the material which affects the passage of water beneath the structure.

Size of screen.	Percentage passing.
1-in.	100.00
¾-in.	94.37
½-in.	89.83
⅜-in.	84.30
No. 4 mesh	71.21
No. 8 "	57.05
No. 14 "	39.93
No. 28 "	21.65
No. 48 "	2.68
No. 100 "	0.37

Readings were taken in the pipes at intervals during the filling of the reservoir and after the full head had been raised, the last one having been made in April, 1926. (Fig. 23(*c*).) Results obtained only in Bays 3, 4, and 5 are considered herein, although other observations were made. Table 5 shows the results obtained. For the sake of comparison, the tabulation is made in a manner similar to that used by the author in Table 1.* The nomenclature is somewhat different, but will be apparent from a study of Fig. 23. In studying Table 5, the first four groups of readings should be considered together. They represent conditions with wide variations of head, before the silt had accumulated over the bottom of the reservoir. The derived values for these groups, shown in Columns (10) to (19), inclusive, seem very consistent except for the values of the percolation factor, C_2. Variations for that factor are not sur-

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 689.

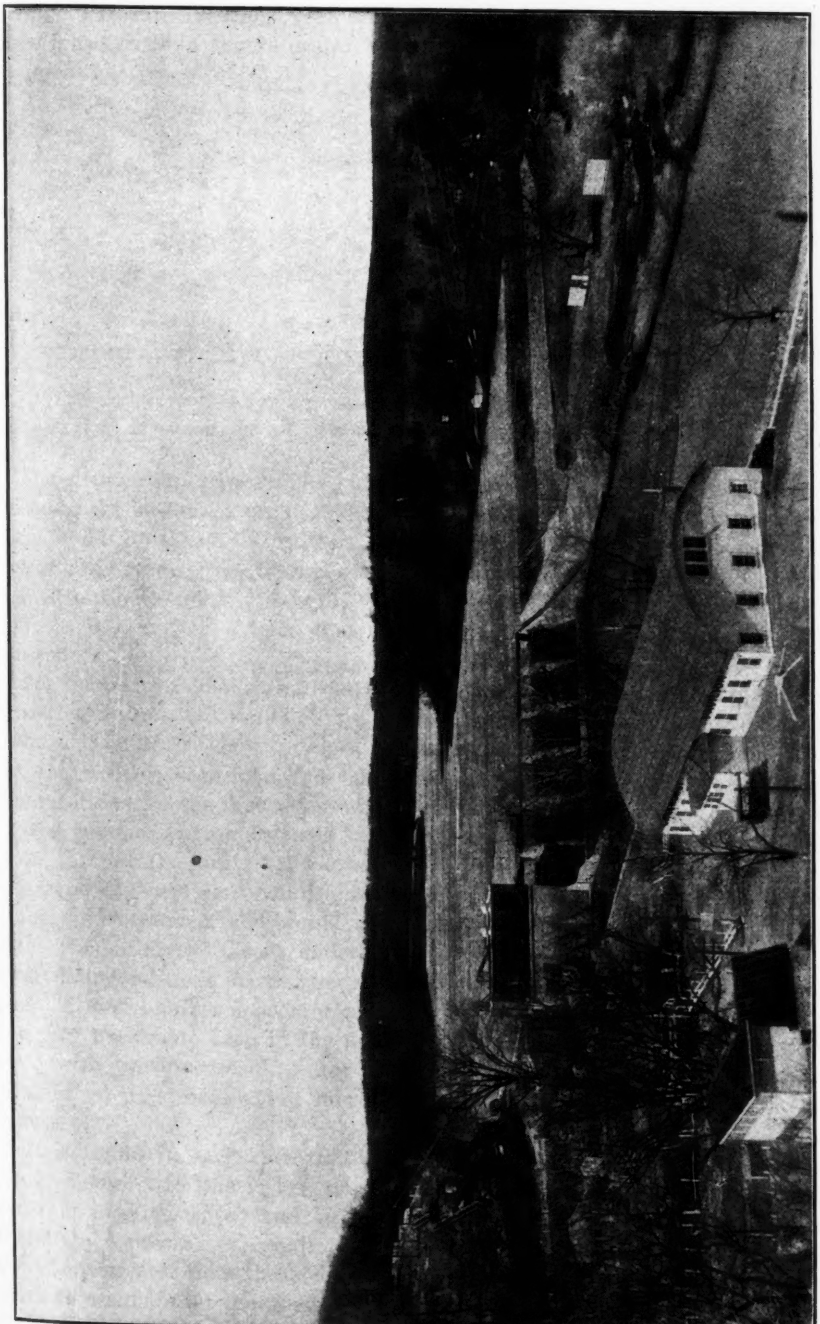

FIG. 21.—VIEW OF PINHOOK DAM AND POWER HOUSE.

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prising. Pressure readings were determined to the nearest 0.1 ft. only, and very slight errors in the values of h_2 would result in large variations in the corresponding values of C_2.

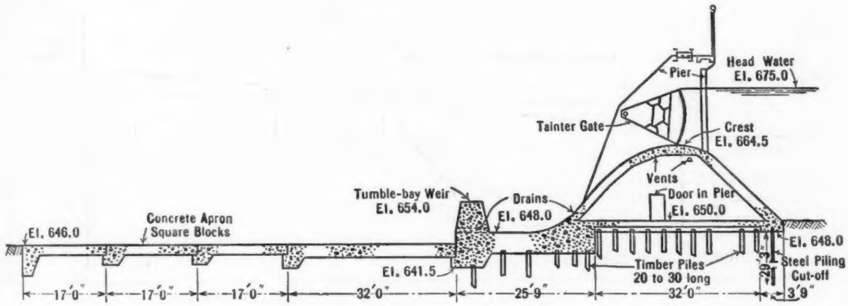

FIG. 22.—CROSS-SECTION OF PINHOOK DAM SPILLWAY, BAYS 2 TO 6.

The fifth group represents conditions under which evidently there was a sealing of the bottom of the reservoir. The results show little relation between actual conditions and theory. In Bays 2 and 3 (Fig. 21), the pressures in Pipe A are actually less than the corresponding pressures in Pipes B and C. Quantities for Columns (15), (16), and (18), of Table 5, are not shown for this group as it is not considered that they would have any value under the circumstances.

For the purpose of making comparison between the pressures found and the theoretical values that would obtain if the "line-of-creep" theory were true, the curves of Fig. 23 are shown. These are based on readings given in Table 5 for Bay 4. It will be seen from this table that the results for this Bay are typical. Computations for the theoretical piezometric line are based on the assumption of free communication between tail-water and the under side of the block apron, through the joint at the down-stream line of the tumble-bay weir. Figs. 23(a) and (b) represent conditions before any appreciable silting of the lake bottom had occurred, and would seem to offer excellent evidence in support of the "line-of-creep" theory under considerable variation of head. Fig. 23(c) is based on conditions existing more than two years after the completion of the dam. A mud blanket approximately 2 ft. deep had been deposited up-stream. The very noticeable drop in the pressures for Pipes A, B, C, and D, as compared with previous readings for the same pipes, is perfect evidence of the value of an up-stream apron in reducing the upward pressure under such a structure.

It is found in each case that the inclination with the horizontal of the actual piezometric line is less than that of the theoretical line as computed, and that in Figs. 23(a) and (b), the two lines cross between the test pipes, B and C. A possible explanation for the fact that the pressures in Pipes C, D, and E are greater than those computed theoretically, might be that it is erroneous to assume that the theoretical line meets the tail-water elevation at the down-stream face of the tumble-bay weir. If it should be assumed that the concrete block apron confines the flow sufficiently to cause upward pressure under

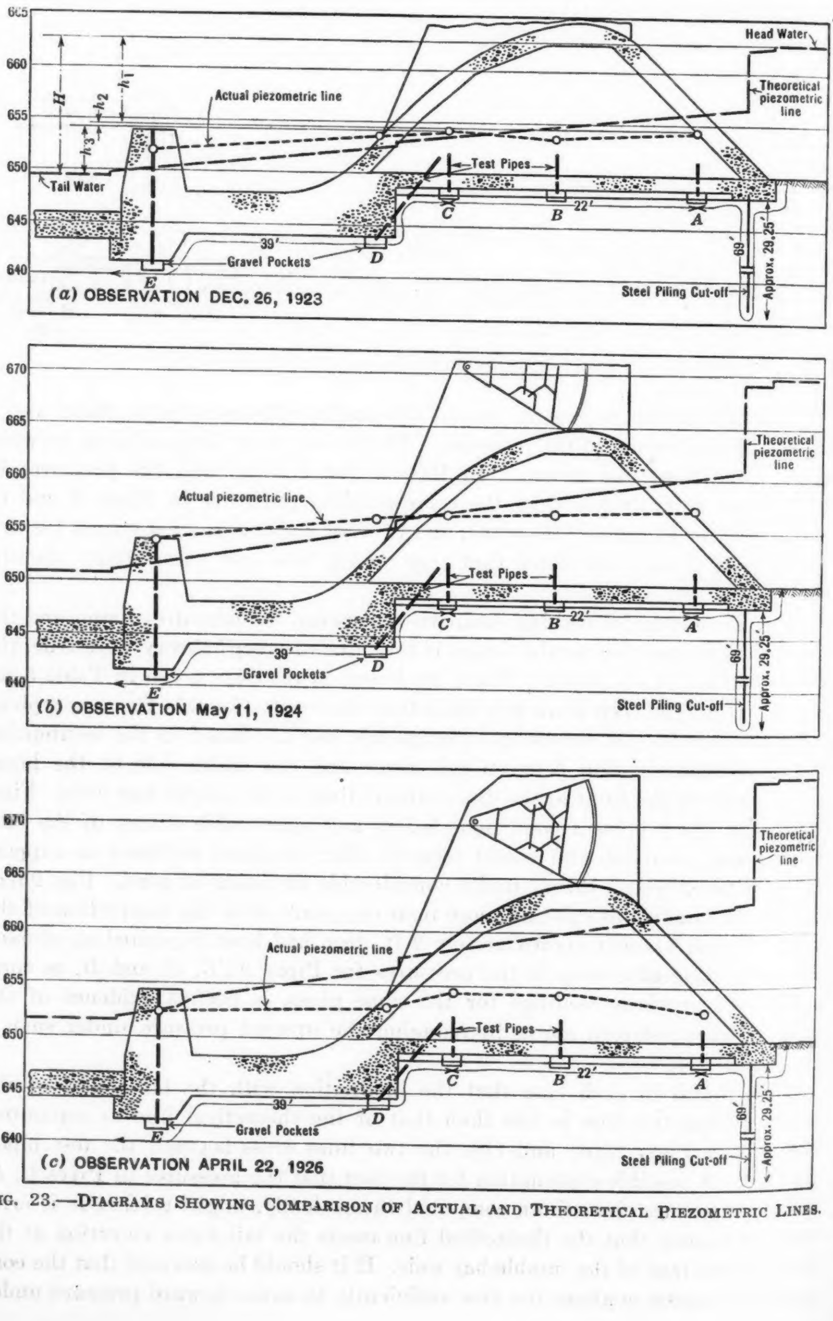

FIG. 23.—DIAGRAMS SHOWING COMPARISON OF ACTUAL AND THEORETICAL PIEZOMETRIC LINES.

TABLE 5.—MEASURED UPLIFT UNDER PINHOOK DAM.

Date.	Bay.	Head-water.	A.	B.	C.	D.	E.	Tail-water.	h_1, Column (3)—Column (4).	h_2, Column (4)—Column (6).	h_3, Column (6)—Column (9).	H, Column (3)—Column (9).	$\frac{h_1}{H}$.	$\frac{h_2}{H}$.	$\frac{h_3}{H}$.	C_1 $69 \div h_1$	C_2 $22 \div h_2$	C_3 $39 \div h_3$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
12-26-23	3	662.7	654.8	654.3	651.2	652.2	652.6	649.4	7.9	0.6	4.8	13.3	0.59	0.05	0.36	9	37	8
	4‡	62.7	54.4	53.6	54.1	53.7	52.1	49.4	8.3	0.3	4.7	13.3	0.63	0.02	0.35	8	73	8
	5	62.7	54.3	53.3	53.8	*	51.5	49.4	8.4	0.5	4.4	13.3	0.63	0.04	0.33	8	44	9
1-10-24	3	66.3	57.0	*	56.2	55.8	*	49.6	9.3	0.8	6.6	16.7	0.56	0.05	0.39	8	28	6
	4	66.3	56.3	56.1	55.9	55.5	*	49.6	10.0	0.4	6.3	16.7	0.60	0.02	0.38	7	55	6
	5	66.3	56.3	*	55.7	*	*	49.6	10.0	0.6	6.1	16.7	0.60	0.04	0.36	7	37	6
2-11-24	3	72.0	60.5	58.9	58.7	*	*	51.2	11.5	1.8	7.5	20.8	0.55	0.09	0.36	6	12	5
	4	72.0	58.9	58.5	58.4	57.8	*	51.2	13.1	0.5	7.2	20.8	0.63	0.02	0.35	5	44	5
	5	72.0	58.8	58.6	58.1	*	*	51.2	13.2	0.7	6.9	20.8	0.63	0.04	0.33	5	31	6
5-11-24	3	69.8	57.3	57.1	57.3	*	54.0	50.6	12.5	0.0	6.7	19.2	0.65	0.00	0.35	6	8	6
	4§	69.8	57.1	56.8	56.6	56.1	54.0	50.6	13.0	0.5	6.0	19.2	0.68	0.02	0.31	5	44	6.5
	5	69.8	56.9	56.1	56.7	*	53.6	50.6	12.9	0.2	6.1	19.2	0.67	0.01	0.32	5	110	6
4-22-26	3	74.8	52.5	53.8	54.3	52.2	53.0	50.7	22.3	+1.8	3.6	24.1	0.93	†	†	3	†	11
	4‖	74.8	52.8	54.3	54.2	52.0	51.5	50.7	22.0	+1.4	3.5	24.1	0.92	†	†	3	†	11
	5	74.8	53.0	53.8	52.9	51.5	53.0	50.7	21.8	0.1	2.2	24.1	0.90	†	†	3	†	18

* Denotes readings were not obtained because of frozen pipes or other obstructions.

† Denotes quantities not computed because of reversed relations for h_2.

‡ See Fig. 23 (*a*).

§ See Fig. 23 (*b*).

‖ See Fig. 23 (*c*).

it, then it would be necessary to assume some point farther down stream at which the pressure would become zero. Such an assumption would lessen the amount of drop at the steel piling cut-off and would flatten the remaining part of the line, tending to make it approach the slope of the actual line.

A second assumption would be that, although the results afford a good substantiation of the theory, it is not strictly true that all the flow follows down the up-stream side of the steel piling and up the other side. It is probable that some of the flow diffuses through the pervious sub-strata, following a course leading diagonally upward from the bottom of the piling, and reaching the under side of the concrete farther down-stream. This would help to account for the comparatively lower pressures in Pipes *A* and *B*. Such would probably be the case in any pervious material of uniform structure, and would certainly be true if the piling extends through or into a stratum more open in nature than the overlying material.

An unfortunate feature of the Pinhook installation is the fact that no readings can be taken during a period of high tail-water because the water level under the spillway deck communicates freely with the tail-water through drain holes and when the tail-water rises it becomes impossible to reach the test pipes.

The author attributes* the rather irregular nature of the results obtained from readings on the Colorado River Dam to the fact that while water enters beneath the dam through a considerable area up stream, it "can escape in appreciable quantity only through the narrow gap of open gravel under Pier *B*", and that "after passing the restricted open gravel area under the cut-off, the flow is free to spread laterally, thus reducing the resistance to outflow". This is very probably the case. A fairly analogous condition was created artificially in a series of tests conducted by E. W. Lane, M. Am. Soc. C. E., and the writer, on the Island Park Dam at Dayton, Ohio.†

Island Park Dam is part of the park system of the City of Dayton. It is a gravity type, concrete, overflow structure, across the Miami River, developing a head of 7 ft. at low-water flow. Conditions are very similar to those shown in Fig. 9,‡ except that the timber sheet-piling is at the up-stream line of the fore apron, and that one of the test pipes extends into the bed of the pond up stream. Tests were made after a mud blanket had sealed the bottom of the pond, and in every instance the readings were far below the theoretical pressures based on the "line-of-creep" theory. An attempt was then made to approximate conditions which existed before the silt deposit had formed. This was done by removing the mud from a small area in the vicinity of the upstream pipe. As a result, the pressure in the up-stream pipe increased to nearly the full theoretical head, although there was slight change in any of the others. This was, no doubt, due to the diffusion of flow after passing through the comparatively small opening near the pipe intake.

* *Proceedings,* Am. Soc. C. E., March, 1928, Papers and Discussions, pp. 689–690.
† *Engineering News-Record,* May 20, 1920.
‡ *Proceedings,* Am. Soc. C. E., March, 1928, Papers and Discussions, p. 696.

The foregoing experiments indicate that, for the design of structures on sand and gravel foundations, it is not wise to assume pressures less than those that apply under the "line-of-creep" theory, although that theory probably errs somewhat on the side of safety. With the depositing of silt over the bottom of a reservoir, the factor of safety increases very materially. As the author indicates, in the case of a low structure, this margin of safety may be diminished in a time of high water if velocities are developed high enough to disturb the bottom up stream from the dam. In a case where no appreciable disturbance of the bottom will occur, it might be reasonable to assume less than the total amount of upward pressure, if the pond can be filled by easy stages and the bottom can be allowed time for partial sealing before the full head accumulates behind the dam. For economic reasons, however, it is generally desired to develop the full head as soon as possible, and such assumptions might lead to trouble.

Installations of apparatus of the nature described by the author can be accomplished at very little cost at the time of construction, and are advocated as serving two worthy purposes. The value of data obtained as a basis for theory and design is obvious. Furthermore, periodic observations will either give assurance of continued stability, or may indicate extraordinary conditions developing under a structure in time to avert serious damage or even failure.

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MEMOIRS OF DECEASED MEMBERS

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JOHN ERNST ERICSON, M. Am. Soc. C. E.*

Died April 16, 1927.

John Ernst Ericson was born in Upland, Sweden, on October 21, 1858, the son of Anders and Sophia (Lind) Ericson. He received his early education in the common schools, and in 1872 he entered the High School and College at Upsala, Sweden, which he attended until 1876, when he entered the Royal Polytechnic Institute at Stockholm, Sweden. He was graduated from this institution with the degree of Civil Engineer in 1880.

Mr. Ericson came to the United States in 1881, his first post of importance being that of Resident Engineer with the Toledo, Cincinnati and St. Louis Railroad Company, with headquarters at Cowden, Ill. He retained this position until the summer of 1882, when he accepted that of Bridge Designer with Hopkins and Company of St. Louis, Mo. In 1883, he was appointed as Assistant on Government surveys in connection with the proposed enlargement of the Illinois and Mississippi River Canal and the construction of the Hennepin Canal. He had an important part in the making of the surveys on these projects as well as some share in outlining the entire plan.

In 1884 and during a part of 1885, he was a Draftsman in the Water Department of the City of Chicago, Ill. He became Assistant City Engineer of Chicago in 1885, and retained this position until 1889, when he was selected as Assistant Chief Engineer of the City of Seattle, Wash., to aid in developing the new gravity water-works system, which was then about to be built there.

In 1890, the Sanitary District of Chicago, the corporation which has control and supervision of the great Drainage Canal System, claimed the services of Mr. Ericson. He remained with the District until 1892, when he became Assistant Engineer in the Bureau of Engineering. In 1893, he was appointed First Assistant City Engineer of Chicago, and in 1897 was elevated to the post of City Engineer, which position he held until his death.

As First Assistant City Engineer and as City Engineer he was in charge of the design and construction of all additions to the water supply system of the city, projects involving the outlay of millions of dollars to provide water for the second greatest city in the United States.

As City Engineer, Mr. Ericson was also in charge of all bridge design, construction, and operation, and was called upon to give expert opinion on a multiplicity of engineering subjects connected with the many city betterments which are taken up every year by the City of Chicago in order to care for the increased business and living facilities necessitated by the rapidly

* Memoir prepared by Loren D. Gayton, Assoc. M. Am. Soc. C. E.

spreading metropolis. Mr. Ericson had exceptional opportunities for experiments to determine the elements of flow of water in large tunnels, and presented an extensive treatise* on this subject to the Western Society of Engineers in 1911, for which he received the Society's medal. He also published other papers and reports on water-works, paving, harbors, subways, etc., among which may be mentioned the following: "The Water Supply System of Chicago, Its Past, Present and Future"; "Report on Transportation Subways for Chicago"; "Report on Creosote Block Pavements"; and "Report on Public Water-Works".

He was President of the Swedish Engineers' Society of Chicago, and a member of the American Society of Mechanical Engineers, Western Society of Engineers, and the American Water Works Association.

Mr. Ericson was recognized as one of the leading authorities in the United States on city betterment, and as an engineer who successfully solved many of the great obstacles that beset the larger municipalities in devising systems of caring for their giant populations. Numerous structures and edifices stand to-day as monuments to his engineering ability.

He died on April 16, 1927, at the Presbyterian Hospital, in Chicago, after undergoing an operation. He is survived by his widow, Mrs. Esther Elizabeth Ericson, and a daughter, Mrs. Ralph Haven Quinlan.

Mr. Ericson was elected a Member of the American Society of Civil Engineers on May 7, 1902.

THOMAS JAMES McMINN, M. Am. Soc. C. E.†

Died September 21, 1927.

Thomas James McMinn was born at Port Hope, Ont., Canada, on October 8, 1854. In 1874 he began his technical work on the construction of the new water-works for Toronto, Ont., Canada, under the late P. A. Peterson, M. Am. Soc. C. E. He remained on this work and its various subsequent extensions, until 1879, when he was made Assistant Engineer to the late R. J. Brough, M. Am. Soc. C. E.

The following year he was engaged in making surveys, plans, specifications, and estimates for the construction of a 6-ft. oak conduit, to be built and extended for a considerable distance into Lake Ontario. In 1881, he was appointed Resident Engineer on this "lake extension", and served in this capacity during 1881 and 1882.

From 1882 to 1889, Mr. McMinn was employed as Assistant Engineer and Draftsman in the Water-Works Department of Toronto. During this period he made a barometric survey of the country extending to the north of Toronto, in connection with a proposed gravitation supply for the locality.

On leaving the Water-Works Department, he came to the United States and entered the employ of the late Rudolph Hering, M. Am. Soc. C. E., who

* "Investigations of Flow in Brick Lined Conduits," *Journal*, Western Soc. of Engrs. Vol. XVI, No. 8 (October, 1911), p. 657.

† Memoir prepared from information on file at Society Headquarters.

was engaged in the private practice of engineering, in New York, N. Y. Mr. McMinn remained with Mr. Hering until January, 1898, when he was elected Assistant Secretary of the Society, which position he retained until his retirement in 1919. Shortly afterward he removed to Bridgeport, Conn., and then to Philadelphia, Pa., where he died on September 21, 1927. During this time he was engaged in editing several technical books for the late Mr. Hering and others.

Such, in brief, is an outline record of a useful life. To most members—and Mr. McMinn was widely known in the Society—his most valuable work was comprised within his official term as Assistant Secretary. The duties of this office were varied, especially in the early days, when the smallness of the organization demanded a wide variety of service of each officer. Always, however, Mr. McMinn gave his main efforts to publications.

At one time he handled the many phases of this work—editing, attending to engraving, proof-reading, and supervising the printing. Later, however, as the work expanded, he devoted his time mainly to the editing. In this he was, happily, successful. Endowed naturally with a clear perception of the fitness of expression, he was at once painstaking, well read, and widely versed in engineering methods. Many of the details of style he instituted are still standard practice, to the lasting credit and reputation of the publications.

Throughout his life his interest in these publications never flagged. A year before he died he went over a number of *Proceedings* in detail and submitted a long list of suggested improvements in its grammatical form—revisions that none but his practiced and discerning eye would have noted. A similar trait of character was noticeable in his reading. Much of his library came to the Society; many of his books, especially the engineering volumes, showed traces of his critical taste in apt marginal notations of improvement in style.

During so many years' intimate connection with the Society journals, he gained a wide and close acquaintance among the members. Endowed with a mildness of manner and a gentlemanly bearing, his relations were always pleasant. No one could exchange ideas with him anent engineering matters, especially the details of rhetorical expression, without feeling his innate grasp of such intricacies.

His qualities of person and character endeared him to his many colleagues at Society Headquarters. He had made for himself a solid place in their esteem. Apparently, the feeling was mutual; the several years after his retirement were punctuated by frequent visits to New York, at which times he always made it a point to keep his many friendships in good repair.

Mr. McMinn was essentially a literary editor and an excellent one. But he was also an engineer; and, above all, a man. His lengthening years of Society work only served to ennoble his character, to bring into stronger relief the qualities of mind and of person that made him revered and loved. His place in the annals of the Society is secure.

Mr. McMinn was married on June 4, 1884, to Ada Jeannette Petman, of Toronto, Ont., Canada, who survives him, together with four children, George,

Stanley P., Mrs. Lowell Grossman, and Mrs. W. C. Hooven. His home life was ever an inspiration to his friends, and his family indeed lived "to call him blessed".

Mr. McMinn was elected a Member of the American Society of Civil Engineers on March 5, 1890. He was also a member of the Engineering Institute of Canada.

LEVI LOCKWOOD WHEELER, M. Am. Soc. C. E.*

Died March 13, 1927.

Levi Lockwood Wheeler was born on a farm near Jackson, Mich., on February 5, 1851, the only son of four children, of Sarah Houseman and Lorenzo Dow Wheeler. After receiving his primary education at a country school, he was graduated from the High School at Jackson, in 1870, and, subsequently, entered the University of Michigan at Ann Arbor, Mich. He completed the regular four-year course in Civil Engineering, and received the degree of Civil Engineer in June, 1874.

Mr. Wheeler entered the Civil Service of the United States on May 8, 1874, as Sub-Assistant on the United States Lake Survey at Detroit, Mich., where, with the exception of one summer, he continued until April, 1881. While on this work he ran one of two lines of precise levels for determining the elevations of the Great Lakes above sea level and, later, he reduced the data relating to this subject and prepared them for publication.† During his connection with the U. S. Lake Survey, Mr. Wheeler aided in the comparisons of standards of length, in the determination of latitudes and longitudes, and in the reduction of the notes relating thereto.

During the summer of 1881, he worked on his father's farm in Jackson County, Michigan.

In February, 1882, Mr. Wheeler was appointed an Assistant Engineer with the Mississippi River Commission at St. Louis, Mo., and remained in this position for six years. He made the topographic and hydrographic surveys of the Mississippi River and adjacent territory from Vicksburg to Natchez, Miss., and for a reach 110 miles long above Memphis, Tenn. He revised the methods which had been in use on precise level work under the Commission, pointed out the errors in methods formerly used, and prepared instructions for field work, which were subsequently followed by the Commission and used on other important surveys in the United States. Mr. Wheeler also had charge of the Computing Division for a number of years and prepared data for the use of the Commission and for publication.

In September, 1888, he was placed in charge of a Government survey for a 14-ft. waterway from Lake Michigan to the Illinois River. He made sur-

* Memoir prepared by John W. Woermann, M. Am. Soc. C. E.

† *Professional Papers No. 24*, Corps of Engrs., U. S. A., Chapter XXII.

veys covering two locations from Lake Michigan to Joliet, Ill., the report and maps of which were printed and used by the Sanitary District of Chicago, Ill., in the construction of the main canal and the Calumet-Sag Canal.

In November, 1890, Mr. Wheeler was appointed United States Assistant Engineer in charge of the location and construction of the Illinois and Mississippi Canal, extending from the Illinois River, near Hennepin, to the Mississippi River, near Rock Island, Ill. The summit level of the canal is supplied by a Feeder carrying the water from Rock River at Sterling, Ill. The main line and Feeder have a surface width of 80 ft. and a depth of 7 ft., the main canal being 75 miles long and the Feeder 29 miles long. Mr. Wheeler was in immediate charge of the Western Section of the main canal which is 27 miles long, and also of the entire Feeder.

The structures on the Illinois and Mississippi Canal (better known as the Hennepin Canal) include 34 locks, 9 aqueduct bridges, 9 railway bridges, 69 highway bridges, 60 culverts, 3 dams, 51 houses, and numerous smaller structures. Construction was commenced in 1892, and the water was turned into the canal on October 24, 1907. Mr. Wheeler was subsequently in charge of the operation and maintenance of the canal until his retirement in 1921. A large number of the structures were designed as well as built under his direction and about $4 200 000 was expended on this canal under his immediate supervision.

In addition to his Government work, Mr. Wheeler was engaged in private practice, mainly in hydraulic work, and as Consulting Engineer and expert witness in connection with a number of projects. He designed and supervised the construction of water-power dams across the Rock River, at Sterling, Dixon, Oregon, and Sears, Ill., two of which were crib dams and two, concrete dams. He also designed a dam for Grand Detour, Ill., which was not built.

Mr. Wheeler was retired from the Government Service on February 5, 1921, at the age of 70 years, the retiring age prescribed by Congress for the Civil Service. On April 5 he returned home from a visit to his son in the Hawaiian Islands, and on April 15 suffered a cerebral hemorrhage, from which he never recovered. Eleven months before his death, he fell in his room, breaking a hip, as a result of which he was confined to his bed until his death on March 13, 1927. His daughter, Mrs. Clingan, states that "Never, in all the six years of his illness did he complain, but was ever thoughtful of others, and wanting others to be happy."

Mr. Wheeler was a great lover of Nature in all her aspects. He loved to fish and hunt, and was well posted on fish, birds, animals, trees, and flowers. He always enjoyed working in his garden and took pride in the beauty and variety of his fruits and flowers, especially of his tulips and irises. He was also an authority on bees and kept a number of hives in connection with his garden.

Extremely thorough, methodical and accurate in all his work, Mr. Wheeler was satisfied with nothing less from his subordinates. In connection with the construction of the Hennepin Canal he established a record for economy and efficiency which seldom has been equalled. He was gifted with a mind and

physique which made it possible for him to keep in contact with all the details of his work, as well as with its larger and broader features. He seemed to be equally at home in the field, on location and construction, and in the office, in computing and designing. He was a man of fine appearance, possessed of remarkable energy and unusual capacity for hard work.

On January 7, 1880, Mr. Wheeler was married, at Detroit, to Isabella Chambers, who died on January 16, 1912. He is survived by four children, Arthur Chambers Wheeler, M. Am. Soc. C. E., of Hilo, Hawaii; Frank Dow Wheeler, Civil Engineer, of Chicago; Mabel Alice Wheeler, of Detroit; and Mrs. Grace Wheeler Clingan, of Sterling; also, one grandchild, Lee Grant Wheeler, of Hilo, Hawaii. Mr. Wheeler was greatly devoted to his family, always keenly interested in their happiness and welfare, and never more happy than when spending his evenings at home surrounded by them.

He was a member of the Benevolent and Protective Order of Elks, at the rooms of which, he was a frequent attendant after his wife's death, and was at one time President of the Sterling Club, a social organization. For a number of years prior to his illness, he had been a Trustee of the First Presbyterian Church of Sterling, from which he was buried.

Mr. Wheeler was elected a Member of the American Society of Civil Engineers on June 4, 1884.

GUY FREDERIC HOSMER, Assoc. M. Am. Soc. C. E.*

Died January 27, 1928.

Guy Frederic Hosmer, the son of Nathaniel M. and Mary A. (Howe) Hosmer, was born in Shrewsbury, Mass., on March 14, 1877.

He was graduated from the Shrewsbury High School in 1892, at the age of fifteen. In 1894, he had his first engineering experience when he assisted Mr. Romeo E. Allen, of Shrewsbury, on the survey of the Old Post Road. The following year Mr. Hosmer became associated with the engineering office of Shedd and Sarle, of Worcester, Mass., and Providence, R. I. When this firm dissolved partnership, he remained in the office of O. Perry Sarle, M. Am. Soc. C. E., of Providence, with which he was connected for many years.

In this position, Mr. Hosmer had a varied experience, including general surveying, oyster bed surveying, triangulation, design and construction of water-works, sewerage systems, wharves, dams, mill buildings, power houses, and the taking of stream flow measurements. From 1903 to 1905, he was in charge of the preliminary surveys and construction for a system of water-works and sewers, including a pumping station, stand-pipe and sand filtration plant, for Hickory, N. C. During 1906 and 1907 he was in charge of the design and construction of brick and iron buildings, with reinforced concrete floors and columns, for the Penobscot Chemical Fibre Company at Great Works, Me., and, in 1908, Engineer in charge of extensive improvements in the water power system at the D. Goff and Sons Mill, at Pawtucket, R. I.

In 1913 and 1914, Mr. Hosmer supervised the building of a sewage disposal plant, at Central Falls, R. I., which involved the construction of an

* Memoir prepared by Percy W. Sarle, Esq., Rumford, Me.

Imhoff tank and sprinkling filters. In 1914 and 1915, he was in charge of the field work for preliminary plans of a $1 000 000 water-works plant for the Town of Warwick, R. I.

Mr. Hosmer remained in Mr. Sarle's office until January 1, 1917, when he accepted a position as Engineer of the Maine Coated Paper Company, at Rumford, Me. In 1921, this Company was consolidated with the Oxford Paper Company, and Mr. Hosmer became Assistant to the General Engineer, which position he retained until his death. As such, he was in charge of the design and erection of new buildings and the alteration of old ones, as well as the laying out and installation of new equipment. He also designed and invented various improvements which were added to the machinery in the Coating Mill. While in this position, he had charge, in 1922, of the remodeling of the plant of the Nashwaak Pulp and Paper Company, at St. John, N. B., Canada.

Mr. Hosmer was very active in community affairs. He was President of the Rumford Mechanics Institute; a member of the Rumford Board of Selectmen; one of the promoters, as well as Director and Chairman of the Greens Committee of the Oakdale Country Club; Chairman of the 1927 Rumford Winter Carnival Committee; and a member of the Building Committee of the Rumford Community Hospital, and also of the Methodist Episcopal Church.

He was a member of the Association of Professional Engineers of the Province of New Brunswick, Canada, and the Bangor, Me., Lodge of the Benevolent and Protective Order of Elks.

Mr. Hosmer had a very keen and analytical mind and was ever ready to solve problems of his own and his associates. He was always greatly interested in new engineering ventures and kept himself informed as to the latest developments in the Engineering Profession.

The greatest tribute that can be paid him is that all his associates, within the organizations he served, or individuals of other organizations with whom he came in contact, are proud that they knew him.

In every position, industrial or civic, which he held, Mr. Hosmer was unsparing in his efforts, unusually efficient, and faithful to every detail. His work was always well done.

He was never married, and is survived by three brothers, William H. and Arthur C. Hosmer, of Camden, Me., and Harry P. Hosmer, of Boston, Mass.

Mr. Hosmer was elected an Associate Member of the American Society of Civil Engineers on November 6, 1907.

REUBEN BENJAMIN SLEIGHT, Assoc. M. Am. C. E.*

Died November 14, 1927.

Reuben Benjamin Sleight, the son of Levi J. and Katherine C. (Buchler) Sleight, was born at Laingsburg, Mich., on June 30, 1889. He received his

* Memoir prepared by R. L. Parshall, Assoc. M. Am. Soc. C. E.

early education in the public schools of his native State, and in 1908 entered the University of Michigan, where he studied engineering for three years.

In June, 1911, Mr. Sleight was employed by the Racine Boat Manufacturing Company, at Muskegon, Mich., as a Draftsman in the mechanical and structural design of lightships for the Federal Government. The opportunities of the West attracted him to Denver, Colo., where he entered the office of the Field, Fellows, and Hinderlider Engineering Company, in September, 1912. His experience with this firm was in designing, drafting, and field surveys, for the Parkman Irrigation District of Wyoming and the Cherry Creek protective work, at Denver.

In January, 1913, Mr. Sleight left this work and was employed immediately as Construction Foreman and Concrete Inspector on irrigation structures and well development for the Tucson Farms Company, near Tucson, Ariz. In June he was transferred to a new line of work, a duty-of-water investigation in the Mesilla Valley, New Mexico. This study under co-operation between what is now the Division of Agricultural Engineering of the United States Department of Agriculture, the New Mexico Agricultural Experiment Station, and the farmers of Mesilla Valley, consisted in the design of gas and electric power pumping plants, their installation and testing,* and assistance in tests on submerged orifices, the results of which have been published.† Mr. Sleight's work in New Mexico was under the immediate direction of F. L. Bixby, M. Am. Soc. C. E.

On June 2, 1913, Mr. Sleight was appointed Assistant Irrigation Engineer in the U. S. Department of Agriculture, which position he held until he entered the Army in 1917. During the winter of 1913-14, he was assigned to the Headquarters Office of the Division of Agricultural Engineering at Washington, D. C., as an Assistant to F. C. Scobey, M. Am. Soc. C. E., on hydraulic engineering research. Subsequently he returned to his former work in New Mexico.

In September, 1914, on leave of absence from his Government position, Mr. Sleight re-entered the University of Michigan from which he was graduated as a Bachelor in Marine Engineering in the early summer of 1915. During his Senior year at the University, he was made an Assistant in the Department of Civil Engineering. On leaving the University and resuming his Government connection, in July, 1915, he was assigned to the work of establishing a research laboratory, at Denver, for the purpose of studying the evaporation from free water surfaces, soils, and river-bed materials. The results of these studies, which were started early in the season of 1916, were published under the title, "Evaporation from the Surfaces of Water and River-Bed Materials."‡ This discussion on a subject of great interest to the Engineering Profession was a comprehensive and authoritative scientific contribution of high value, and immediately attracted wide attention. Of the many

* The report of tests on turbine centrifugal pumps was published in *Journal of Agricultural Research*, Vol. XXXI, No. 3.

† "Research Studies on the Flow of Water in Open Channels Carrying Heavily Silted Water," *Bulletin 97*, New Mexico Agricultural Experimental Station.

‡ *Journal of Agricultural Research*, Vol. X, No. 5, July 30, 1917.

manuscripts prepared by Mr. Sleight, it is thought his summary of the work on evaporation was the most outstanding.

Mr. Sleight's military career began in October, 1917, with his appointment as Production Engineer, Signal Corps, U. S. Army. In this capacity he directed his efforts to the compilation of production statistics until January 16, 1918, when he was commissioned as Second Lieutenant in the Signal Reserve Corps. He was promoted to First Lieutenant, Air Service, U. S. Army, on September 25, 1918, and made Tonnage Officer in charge of compiling estimates for the General Staff and the American Expeditionary Force, of tonnage of all air service material to be carried overseas. He remained in the Air Service until July, 1919, but had no foreign experience. He held the commission of Captain in the Air Service Officers' Reserve Corps until his death.

In September, 1919, Mr. Sleight accepted a position as Appraisal Engineer with H. E. Riggs, M. Am. Soc. C. E., at Ann Arbor. While with Professor Riggs, he assisted in the appraisals of the Detroit United Railways; the Kentucky and West Virginia Power Company; the Northern Ohio Light and Power Company; the Columbus Power Company; the Columbus Railroad Company; and the Gas Light Company, all of Columbus, Ga., as well as the Augusta-Aiken Railway and Electric Corporation. In this appraisal work, he was, first, a Field Inspector, then in charge of field investigations and, later, assigned to office engineering.

He severed his connection with the appraisal work at Ann Arbor in 1922 to take the position of Engineer with the Minnesota Tax Commission. An arrangement was made whereby the College of Engineering and Architecture of the University of Minnesota supervised the valuation of public utilities, and, occasionally, of other properties for the Tax Commission. The Engineering Department of the Commission was thus established, and Mr. Sleight was engaged to do the appraisal work. He placed the Department on an efficient and effective basis and was most highly thought of by all who came in contact with him during the five years he spent in Minnesota.

Because of his ambition and earnestness of purpose, he was recognized as an outstanding leader in engineering and scientific circles. His proficiency as a scholar gained for him a membership in the University of Michigan Chapter of Sigma Xi in 1915, while yet an undergraduate.

Mr. Sleight was a registered Civil Engineer of the State of Michigan, and a member of the Engineers' Club of Minneapolis. He was the author of a number of papers published in various engineering and scientific periodicals, and in 1927 wrote an important paper on Engineering Economics which, in a National contest, gained such recognition as to result in his appointment on the Engineering Staff of the Federal Department of Commerce as an Aide to the Secretary, Herbert Hoover, Hon. M. Am. Soc. C. E.

As an Assistant to Secretary Hoover, he was detailed to work out special waterways problems. At the time of his death, on November 14, 1927, he was in Vermont, having left Washington in an airplane to gather advance information for Mr. Hoover in the areas then flooded. The injuries from which he died were suffered in the landing of the plane at Montpelier, Vt.

He was married on October 30, 1913, to Doris M. Cutter, of Muskegon, Mich., who survives him.

By nature modest and retiring, Mr. Sleight's manner was tactful and most courteous. He had the power to command loyal service and form sincere friendships, and those who knew him well were continuously gratified by his successful advancement in his chosen life work.

Mr. Bixby, under whom he was engaged in New Mexico, states that Mr. Sleight's work was of the highest order, and that he possessed to a high degree the ability to combine an understanding of the technique of engineering investigations with the practical application of theory, while his pleasant personality and earnest willingness to co-operate with others in the solving of difficult engineering problems, gave promise of a successful career in scientific research. Moreover, his affability did not prevent his strict and conscientious attention to business when on duty, although his company in leisure hours was most enjoyable. Mr. Bixby feels that in the death of this young man the Engineering Profession suffers an inestimable loss.

Mr. Sleight was elected a Junior of the American Society of Civil Engineers on December 6, 1915, and an Associate Member on April 25, 1921.